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Precast Concrete Panel Thickness for Epoxy-Coated Prestressing Strands

Final Report
Volume 1—Technical Report

December 1994

Sponsored by the
Iowa Department of Transportation Highway Division,
and the Iowa Highway Research Board

Iowa DOT Project HR-353
ISU-ERI-Ames-95066

Iowa Department of Transportation

report

College of Engineering
Iowa State University

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ABSTRACT

This final report for Phase 1 of the research on epoxy-coated, prestressing strands in precast prestressed concrete (PC) panels has been published in two volumes. Volume 1--Technical Report contains the problem description, literature review, and survey results; descriptions of the test specimens, experimental tests, and analytical models; discussions of the analytical and experimental results; summary, conclusions, and recommendations; list of references; and acknowledgments. Volume 2--Supplemental Report contains additional information in the form of summarized responses to the questionnaires; graphs showing the strand forces; figures showing the geometry of the specimens and concrete crack patterns that formed in the strand transfer length and strand development length specimens; and graphs of the concrete strains in the strand transfer length specimens, load-point deflections, and strand-slip measurements for the strand development length specimens.

PC subdeck panels that act compositely with a cast-in-place reinforced concrete topping slab have been used in PC girder bridges in Iowa for many years. The durability of this alternate form of bridge deck construction has been questioned because the prestressing strands and welded wire fabric (WWF) that reinforce the panels are not epoxy coated. The primary objective of the research was to determine the feasibility of using grit-impregnated, epoxy-coated strands and epoxy-coated WWF in thin PC panels. Since larger bond stresses occur between a grit-impregnated, epoxy-coated strand and the surrounding concrete when compared to uncoated strands, a minimum thickness for a PC panel needed to be established so that concrete cracking would not occur when the panels were prestressed. Other objectives of the research were to determine the transfer and development lengths for 3/8-in. diameter, seven-wire, 270-ksi, low-relaxation, grit-impregnated, epoxy-coated (coated) and bare (uncoated) prestressing strands.

The research program included a comprehensive literature review, surveys of design agencies and precast manufacturers to establish the use of epoxy-coated reinforcement in bridge construction, an extensive laboratory study that involved the testing of 115 PC specimens, and analytical studies of strand transfer and development lengths. The survey responses showed that the use of epoxy-coated strands in bridge components has been minimal. Only one design agency has used coated strands in PC subdeck panels. The experimental testing revealed that when coated strands are located and the middepth of a PC panel, a 3-in. minimum thickness was required to prevent concrete cracking when the strands were cut, and that the measured coated-strand transfer and development lengths were about one-half of those measured lengths for uncoated strands. The amount of concrete side cover provided in the test specimens affected the uncoated-strand transfer and development lengths but apparently did not affect the coated-strand development length. The influence of concrete side cover on the transfer length for coated strands was inconclusive. For the multiple-strand specimens, the 6-in. strand spacing did not appear to affect the transfer or development lengths for either the coated or uncoated strands. Some of the analytical models proposed by other researchers provided a good prediction of the strand transfer and development lengths. The AASHTO Specification expression for strand development length significantly overestimated the measured strand development length for coated strands, substantially underestimated this length for uncoated strands with small amounts of concrete side cover, and provided a good prediction for this length for uncoated strands with adequate concrete side cover and spacing.

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E_{ci} = modulus of elasticity of the concrete when the prestress force is applied to the concrete

E_p = modulus of elasticity of the prestressing strands

E_s = modulus of elasticity of the nonprestressed bar reinforcement

ES = prestress loss due to elastic shortening of the concrete

f'_c = concrete compressive strength at 28 days

f'_{cd} = concrete compressive strength when transverse loads are applied to the member

f'_{ci} = concrete compressive strength when the prestress force is applied to the concrete section

f'_s = ultimate tensile strength of the prestressing strands

f^*_{su} = strand stress associated with the nominal moment strength of the member

f_{cds} = concrete stress at the centroid of the tendons caused by the superimposed permanent dead loads that are applied to the concrete section

f_{cir} = compressive stress at the centroid of the tendons immediately after detensioning of the strands and including dead load stresses associated with the self-weight of the member

f_d = service level dead load stress at the location of the extreme tension fiber caused by the application of the external loads

f_o = stress in nonprestressed steel

f_{pe} = concrete compressive stress which is induced by the effective prestress force, at the location of the extreme tension fiber caused by application of the external loads

f_r = concrete modulus of rupture strength at 28 days

f_{ri} = concrete modulus of rupture strengths when the specimens were prestressed

f_{se} = effective strand prestress

f_{si} = initial stress in the prestressing strands before any losses have occurred

F_o = force in the nonprestressed reinforcement

h = specimen thickness

I_g = gross moment of inertia of the cross section with respect to the axis of bending and without considering the steel reinforcement

L = span length

L_d = strand development length

L_{fb} = strand flexural-bond length

L_t = strand transfer length

M_c = concrete cracking moment

M_{cr} = critical moment

M_g = moment caused by the self-weight of the member

M_{max} = maximum factored level moment

M_n = nominal moment strength

M_s = strand-slip moment

M_u = ultimate moment

P = transverse elastic load

P_c = applied load when the first concrete crack was detected

P_s = applied load for when a strand-slip was equal to 0.01 in.

P_u = ultimate transverse load

PL = prestress loss

RH = mean annual ambient relative humidity expressed in percent

SH = prestress loss due to shrinkage of the concrete

T_e = effective strand force

T_p = strand tension force

U'_d = nondimensionalized bond stress along the plastic portion of the strand flexural-bond length

U'_t = nondimensionalized bond stress along the plastic portion of the strand transfer length

U_d = plastic bond stress along the plastic zone of the strand flexural-bond length

U_t = plastic bond stress along the plastic zone of the strand transfer length

V_c = nominal shear strength provided by the concrete

V_d = service level shear force due to dead loads

V_i = factored level shear force corresponding to the loading condition that produces the maximum factored level moment

V_n = nominal shear strength of a PC beam

V_s = nominal shear strength provided by the web reinforcement

V_u = maximum shear force induced by the maximum transverse load

w_d = uniform self-weight of the PC member

X = distance from the transverse load to the near end of the specimen

Y_t = distance from the centroid of the uncracked cross section without reinforcement to the extreme tension fiber of the cross section when the external loads are applied

β_1 = Whitney Stress Block factor for concrete strength

Δ_p = load-point deflection corresponding to the load P

ϵ_1 = effective prestressing strand strain

ϵ_2 = additional strand strain associated with decompression of the concrete

ϵ_3 = additional strand strain that is equal to the increase in the magnitude of the strain at the end of the concrete decompression stage to the strain that occurs at the nominal moment strength of the member

ϵ_c = maximum concrete strain that is associated with the moment strength M_n

ϵ_m = concrete strain beyond the strand transfer length that was measured immediately after strand release

ϵ_o = strain in the nonprestressed reinforcement that was located near the compression face of the cross section

ϵ_p = total strand strain

ϵ_y = yield strength of the nonprestressed steel

γ^* = factor for type of prestressing steel

ρ^* = prestressing steel ratio

1. INTRODUCTION

1.1. Problem Description

A composite bridge deck consists of a reinforced concrete (RC) topping slab that is cast directly on top of thin precast prestressed concrete (PC) subdeck panels. This type of slab system has been used as an alternative to a monolithic full-depth RC slab on secondary roads in Iowa for many years. Figure 1.1 shows a partial isometric view of a highway bridge that has a composite deck supported by PC girders. The PC subdeck panels, which are reinforced with a row of prestressing strands positioned at the midthickness of a panel and with a layer of welded wire fabric (WWF) layed directly on top of the strands, are designed to replace the lower portion of a full-depth RC deck, including the bottom layer of longitudinal and transverse reinforcement.

Corrosion resistance for the steel reinforcement is commonly required in bridge construction, particularly when the bridge is exposed to environments that contain salts. Presently, the Iowa Department of Transportation (Iowa DOT) requires that all of the standard deformed reinforcing bars used in full-depth RC decks be epoxy-coated to provide corrosion resistance. However, the prestressing strands and WWF used in the PC panels of composite decks have been excluded from the epoxy-coating requirement, since the longitudinal prestressing in the panels minimizes transverse concrete cracks. In the transverse direction, the panels are not prestressed; therefore, longitudinal concrete cracks might develop in some panels leaving the reinforcement potentially vulnerable to corrosion. As evidence of the susceptibility of uncoated reinforcement to corrosion, a field inspection that was performed in 1989 of three composite bridge decks in Hardin County, Iowa, as reported in Ref. [2], revealed numerous hairline cracks located directly below and extending along the length of the uncoated strands in many panels and rust staining on the bottom surface of a few panels.

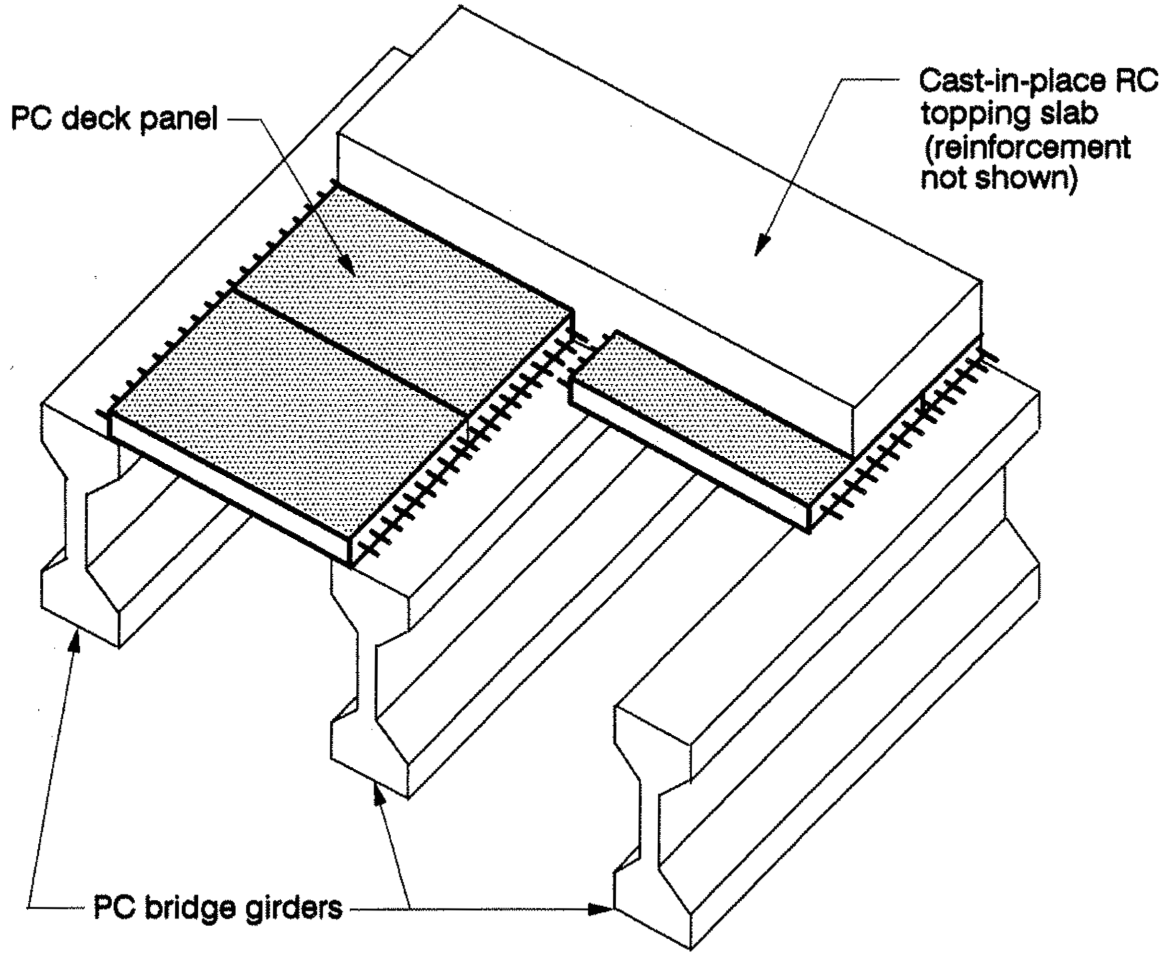


Figure 1.1. Isometric drawing showing PC bridge girders, PC deck panels, and cast-in-place RC topping slab

The feasibility of using grit-impregnated, epoxy-coated prestressing strands and epoxy-coated WWF in PC sub-deck panels for future applications in composite bridge deck construction was the subject of Phase I of the research on coated reinforcement for PC panels. The results of the Phase I research are contained in this report. Presently, the Iowa DOT specifies a minimum thickness of 2.5 in. for panels containing uncoated strands and uncoated WWF. Since researchers [12,15] have reported concrete splitting failures within the strand transfer lengths for 1/2-in. diameter, grit-impregnated, epoxy-coated, single-strand specimens, additional concrete cover over the strands may be required for PC panels containing coated strands.

The primary objective for Phase I of the research was to establish a recommended minimum thickness for PC bridge subdeck panels which contain 3/8-in. diameter, seven-wire, 270-ksi, low-relaxation, grit-impregnated, epoxy-coated, prestressing strands and epoxy-coated WWF to prevent through-thickness concrete cracking of the panels when they are prestressed. For simplicity, this strand description will be termed coated strands, and bare strands of the same size and type will be referred to as uncoated strands in this report. Other objectives of this phase of the research included the establishment of the short-term bond performance of coated strands and the development of the seating characteristics of the wedge-shaped grips that are used for holding coated strands at the anchorage end of a prestressing bed. The research objectives were accomplished by evaluating the strand transfer and development lengths, studying the influence of specimen size and multiple strands on the transfer and development lengths, and measuring the strand displacements at the chucks during prestressing of coated and uncoated strands. For comparative purpose, specimens which contain the same size and type of uncoated strands were constructed and tested.

1.2. Literature Review

Most of the previous research on the bonding characteristics of prestressing strands has been confined to the use of bare strands. In 1959, Hanson and Kaar [20] investigated the development length of seven-wire strands by testing 47 beams prestressed with 1/4, 3/8, or 1/2-in. diameter, 250-ksi, stress-relieved, bare strands that had various embedment lengths. The prestress in the strands before release was between 45% and 59% of their ultimate tensile strength. Strand forces were transferred with a gradual release technique when the concrete compressive strength attained 4500 psi, except for a few specimens for which the strands were released at a 3500 psi concrete strength. The development lengths were found to be 60 in. for 3/8-in. diameter strands and 80 in. for 1/2-in. diameter strands. Kaar and Hanson's research provided the basis for the strand development length equations in the American Concrete Institute (ACI) Building Code Section 12.9.1. [3,4] and in the American Association of State Highway and Transportation Officials (AASHTO) Specification Eq.9-32 [1].

The influence of concrete strength and elapsed time since prestressing on the strand transfer length was investigated by Kaar et al.[25]. They prestressed rectangular specimens with 1/4, 3/8, 1/2 or 0.60-in. diameter, uncoated, stress-relieved strands by flame cutting when the concrete compressive strength was between 1660 and 5000 psi at the time of prestress transfer. Kaar et al. concluded that for this range of concrete strength, the strand transfer length was essentially unaffected. After one year had elapsed since prestressing their specimens, the average strand transfer length for all of the strand sizes had increased by about 6%, and the maximum percent increase in this length was about 19%.

Kaar and Hanson [24] performed bond fatigue tests of pretensioned concrete beams that simulated loadings on PC railroad ties. Their rectangular beams, which contain a single 3/8-in. diameter, 270-ksi, uncoated strand, were tested with cyclic loading applied near the end of the transfer length. The cyclic load, which varied as a sine-shaped function from 10% to 100% of the maximum load, was applied at 250 cycles per minute. The maximum load that was applied to some specimens corresponded to the load that caused a flexural concrete crack to open to a width of 0.001 in. For other specimens, the maximum load that was applied was the load that produced a bending moment equal to1.15 times the concrete-cracking moment of the cross section. Kaar and Hanson concluded that in order to obtain a bond fatigue life of three million load cycles, a severe transverse load, which causes an existing concrete crack to open to more than 0.001 in., must be applied at a location that is not less than 2.2 times the strand transfer length from the end of a specimen. For less severe transfer loads that cause an existing concrete crack to open to less than 0.001 in., the load cannot be placed closer than 1.2 times the strand transfer length from the end of a specimen. These tests also revealed that an early bond failure of a prestressing strand in members subjected to repeated loading was caused by concrete cracks in or near the end of a strand transfer length. The researchers also concluded that the strand surface condition can greatly influence the strand transfer length.

Ban et al. [11] studied the bond characteristics of single 3/8-in. (actually 9.3-mm) diameter, seven-wire, 240-ksi, stress-relieved, uncoated-strand specimens by performing strand pull-in tests. The influence of the strand surface condition (rusted or unrusted), concrete compressive strength at transfer, and magnitude of the prestress on the strand transfer length were investigated. They concluded that the strand surface condition could greatly affect the strand transfer length. The strand transfer length for rusted strands was found to be 33% to 50% shorter than the transfer length for unrusted strands. For their concrete compressive strength range (4068 to 6187 psi) at stress transfer and for a prestress force of 11.83 kips (corresponding to a strand stress of 148 ksi), Ban et al. suggested that the concrete strength had little effect on the strand transfer length.

In 1963, Janney [22] reported on a strand transfer length study that involved 250-ksi and 270-ksi, stress-relieved, prestressing strands. Six 4 1/2-in. wide by 3 1/2-in. high by 8-ft long concrete prisms were prestressed with a 1/2-in. diameter uncoated strand that was positioned at the centroid of the cross section. The effective strand prestress for these specimens was about 163 and 176 ksi for the 250 and 270-ksi strands, respectively, and the concrete compressive strength was about 4100 psi when the strands were released. Two specimens were cast for each of the following strand conditions: 250-ksi strand with a clean and bright surface, 270-ksi strand with a clean and bright surface, and 270-ksi strand with a medium intensity of rust on the surface. For these three strand conditions, the transfer lengths were found to be about 25, 30, and 20 in., respectively. Janney concluded that the difference in the measured strand transfer lengths for the 270-ksi strands and the 250-ksi strands was not significant.

Over and Au [39] investigated the strand transfer lengths for 1/4, 3/8, and 1/2-in. diameter, uncoated, 250-ksi, stress-relieved, prestressing strands. All of their 3-in. square prism specimens were prestressed with a single strand. The concrete compressive strength was listed as 4900, 4180, and 5500, and the initial prestress in the strands was equal to 164, 160, and 170 ksi for the specimens containing 1/4, 3/8, and 1/2-in. diameter strands, respectively. Since the strand stresses were not the same, the researchers applied a linear interpolation to the test data to obtain a strand transfer length corresponding to a 150-ksi stress level in each of the three strand sizes. The resulting strand transfer lengths were 20, 30, and 35 in. for the 1/4, 3/8, and 1/2-in. diameter strands, respectively.

Jones and Furr [23] examined the transfer length of uncoated prestressing strands in PC panel subdecks. The effect of cyclic loading on the strand transfer length and panel stiffness was also studied by testing 20 panels that were 3 1/4-in. thick by 22-in. wide. The specimens were prestressed with either 3/8 or 1/2-in. diameter strands. Both normal-weight and lightweight concrete specimens were cast. A gradual release of the strands was used to induce the prestress force in the panels. Their tests of 3/8-in. diameter, seven-wire strands with a 162-ksi initial strand prestress and a normal weight concrete strength of 5100 psi when the strands were cut, resulted in an average strand transfer length of 22 in. Based on the results for all of their specimens, they concluded that the concrete weight used has little effect on the strand transfer length and that the effect of cyclic loading on strand transfer length and panel stiffness was negligible.

In 1976, Martin and Scott [36] presented a method for designing PC flexural members which are too short to provide an embedment length that will develop the full tensile strength of the strand. To calculate the stress distribution along a prestressing strand due to the application of transverse design loads, the researchers developed bilinear equations for the strand force versus concrete embedment length that were based on the test data reported by Hanson and Kaar [20]. For 1/4 through 0.6-in. diameter strands, Martin and Scott proposed a transfer length of 80 strand diameters that was based on the intersection point of their bilinear equations. The flexural bond lengths for these strand sizes that is implied in their equations were considerably higher than the lengths specified by the ACI Building Code [3].

Based on the results of a test program of 36 pretensioned hollow core units, Anderson and Anderson [5] concluded that the ACI Building Code [3] requirement on the strand development length is adequate if the free-end draw-in of a strand at the time of transfer of the prestress force to the concrete is less than a proposed value calculated from an empirical expression. Free-end draw-in of a strand refers to the amount of strand movement that occurs at the end of the specimen immediately after the concrete member has been prestressed. A free-end draw-in of approximately 0.08 in. was considered to be acceptable for a 3/8-in. diameter, prestressing strand that has an initial prestress equal to three-fourths of its ultimate tensile strength.

In 1977, Zia and Mostafa [45] finished a literature survey of the bond development for prestressing strands. They proposed strand transfer and development length equations which account for the effects of strand size, initial strand prestress, and concrete compressive strengths ranging from 2000 to 8000 psi when the strands are detensioned. Their equation for strand transfer length is more conservative than the ACI Building Code [3] requirement. For a small size strand, the difference between the strand transfer length computed according to the ACI Building Code equation and their expression is quite small; however, for large size strands, especially when the prestress force is transferred at a low concrete strength, the difference in the two computed lengths is large. They stated that the ACI Building Code requirement for flexural-bond length underestimates the actual length according to Hanson and Karr's [20] test results. Zia and Mostafa suggested that a 25% increase in the ACI Building Code flexural-bond length should be applied.

Dorsten et al. [18] reported on the availability and major properties of epoxy-coated, prestressing strands which were manufactured by Florida Wire & Cable Company. They mentioned that the strands have good corrosion resistance and that under normal temperatures, slippage between the strand and the epoxy coating does not occur until the strand reaches its ultimate strength. They reported that two types of epoxy-coated strands are available: smooth-surfaced and grit-impregnated epoxy-coated strands. The smooth-surfaced epoxy-coated strand was developed for unbonded tendons. Tests of strands with this type of coating surface have revealed that minimal bond strength was developed between concrete and the smooth coating. The grit-impregnated, epoxy-coated strand was developed for bonded tendons. The grit protrusions from the coating surface develops the bond between strand and concrete. Dorsten et al. commented that the grit size and concentration of the grit used on the strands manufactured by Florida Wire & Cable Company was selected so that the transfer length of the coated strands was approximately equal to that for the same size of uncoated strand. Because concrete splitting occurred across the width of some of their specimens that contained a single, grit-impregnated, epoxy-coated strand when the strand was released, these researchers suggested that additional concrete cover should be provided when coated strands are used when compared with the cover provided for the same size of uncoated strands.

In October 1988, a memorandum [19] that addressed design criteria involving prestressing strands was issued by the Federal Highway Administration (FHWA) to the Regional Federal Highway Administrators. This document revised the multiplication factors for the AASHTO Specification development length equation (Eq. 9-32) [1], which had been higher in a previous FHWA requirement and prohibited the use of 0.6-in. diameter strands in federally funded projects. The October 1988 memorandum specified that

"(1) The use of 0.6-in. diameter strand in a pretensioned application shall not be allowed;

(2) Minimum strand spacing (center-to-center of strand) will be four times the nominal strand diameter;

(3) Development length for all strand sizes up to and including 9/16-in. special strand shall be determined as 1.6 times AASHTO equation 9-32; and,

(4) Where strand is debonded (blanketed) at the end of a member, and tension at service load is allowed in the precompressed tensile zone, the development length shall be determined as 2.0 times AASHTO equation 9-32, as currently required by AASHTO article 9.27.3.

Exceptions to the above criteria are as follows:

(1) Development length for prestressed piling subjected to flexural loading shall be determined as indicated above. Development length for embedded piling not subjected to flexural loading shall be determined as per AASHTO equation 9-32, and the use of 0.6 inch strand will be allowed.

(2) Development length for pretensioned precast sub-deck panels or precast pretensioned voided deck plank, shall be determined as outlined above, or alternatively, by utilizing AASHTO equation 9-32 for development length and designing and tensioning on the basis of a guaranteed ultimate tensile strength (GUTS) of 250 ksi and release of prestress at 70 percent of GUTS regardless of the type of strand used (i.e., 250 or 270 ksi strand)."

Since May 1989, updates prepared (initially on a quarterly basis) by Susan N. Lane have been issued to provide information about the status of research on the development length of prestressing strands for PC members. In the first issue [26], Lane stated that the October 1988 FHWA memorandum was issued because of technical incompatibilities that existed between the types of strands that were tested for developing the AASHTO Specification Eq. 9-32 and strands that are in use. The AASHTO Specification equation for strand development length was based on test results of 250-ksi, stress-relieved strands which had a stress not exceeding 70% of the ultimate tensile strength immediately after transfer. However, the dominant strand commonly used in construction is a 270-ksi, low-relaxation strand, and the AASHTO Specification allows for a strand stress immediately after prestressing of up to 75% of the ultimate tensile strength of this strand.

Cousins, Johnston, and Zia [12-14] have studied the transfer and development lengths of both uncoated and grit-impregnated, epoxy-coated prestressing strands. Their tests were conducted on rectangular specimens which contained a single strand of 3/8, 1/2 or 0.6-in. diameter. For the 1/2-in. diameter, epoxy-coated strands, three different grit densities for the coating were tested to investigate the effect that the grit density had on the strand transfer and development lengths. This analysis of the test results indicated that the transfer and development lengths for epoxy-coated strands are considerably shorter than those lengths for uncoated strands of the same diameter when the same release stress was used. Also, they concluded that the transfer and development lengths for both epoxy-coated and uncoated prestressing strands increase as the strand diameter increases. For epoxy-coated strands, these lengths decrease as the grit density increases. During their tests, longitudinal concrete splitting occurred in two of their transfer length specimens which were prestressed by an epoxy-coated strand. For some of their specimens that were subjected to fatigue loading, they concluded that the effect of cyclic loads on the strand transfer length, strand development length, and flexural strength of members was not significant. Based on their analytical model representing the mechanics of force transfer between prestressing strands and the surrounding concrete, Cousins et al. [14] proposed equations for transfer and development lengths which could be used for both epoxy-coated and uncoated prestressing strands.

Lane [27-33] reported on an FHWA research project that is addressing the bond performance of prestressing strands. Fifty rectangular specimens that were prestressed with various sizes (3/8, 1/2, and 0.6-in. diameter) of 270-ksi, low-relaxation coated or uncoated strands were tested. The results showed that the strand transfer and development lengths were different for single and multiple- strand specimens. Lane [32] concluded that the AASHTO Specification Section 9.20.2.4 [1] which assumes a strand transfer length of 50 times the nominal strand diameter was found to be conservative for all specimens that contained one epoxy-coated strand or four 0.6-in. diameter, epoxy-coated strands.

Lane also noted that for specimens containing one or four uncoated strands of any diameter or four 3/8 or 1/2-in. diameter epoxy-coated strands, the transfer length of 50 strand diameters underestimates the required strand transfer length. The transfer lengths for uncoated prestressing strands were determined to be about 1.6 times the strand transfer lengths for epoxy-coated strands of the same diameter. All of the specimens that were reinforced with four strands had longer strand transfer lengths than those for the single-strand specimens that had the same strand size and coating. Lane [33] has stated that the AASHTO Specification strand development length equation [AASHTO Eq. (9-32)] is conservative for specimens reinforced with either one uncoated or epoxy-coated prestressing strand or four epoxy-coated strands, while the equation underestimates the required strand development length for specimens containing four uncoated prestressing strands. Phase II of the FHWA research on strand transfer and development lengths will involve bridge girders and subdeck panels that were cast during the summer of 1993.

Transfer and development lengths of 270-ksi, low-relaxation, uncoated prestressing strands in an AASHTO Type I girder were investigated by Deatherage et al. [16,17]. They concluded that the portion of the ACI Building Code [3] strand development length equation, given in ACI Art. 12.9.1 that accounts for the strand transfer length is conservative. However, the total equation which predicts the strand development length underestimates the required length. These researchers have suggested that a 50% increase in the flexural-bond length portion of the ACI Building Code strand development equation should be used. In response to the October 1988 FHWA memorandum, Deatherage et al. have stated that the use of 0.6-in. diameter strands should not have been prohibited and that the strand development length obtained from the AASHTO Specification Eq. 9-32 should not have been increased by as much as 1.6 times for all strand sizes.

Shahawy et al. [43] tested AASHTO Type II girders to establish the transfer and development lengths for uncoated prestressing strands. Their results for the transfer lengths of 1/2 and 0.6-in. diameter strands were 30 and 34 in., respectively, which were slightly longer lengths than those lengths predicted by the AASHTO Specification Article 9.20.2.4 requirement of 50 times the strand diameter. Their measured development lengths for 1/2 and 0.6-in. diameter strands were 40% and 60% longer than the lengths established by the AASHTO Specification Eq. 9-32 [1], respectively. Shahawy et al. noted that the strand development length was greatly influenced by shear and confinement reinforcement.

Lane [27, 29-31] reported that Burns studied the influence of fatigue loading on the behavior of PC bridge girders. The major test variables included 1/2 and 0.6-in. diameter, uncoated prestressing strands and rectangular and I-shaped beam specimens. To evaluate the effects of fatigue, Burns conducted static tests before and after the girders were subjected to cyclic loading. The specimens that were prestressed with the 0.6-in. diameter strands experienced some concrete cracking and spalling when the strands were released by flame cutting [27]. Before any cyclic loads were applied, the measured strand transfer lengths for the 1/2 and 0.6-in. diameter, uncoated strands were between 60 to 70 and 65 to 80 times the nominal strand diameter, respectively, and the strand development lengths [29] were 0.9 and 1.0 times of the lengths predicted by the AASHTO Specification Eq. 9-32 [1], respectively. Development length tests were also performed on six I-shaped girders after they had been subjected to one million cycles of load, with the maximum load equal to the load that caused the concrete to develop tension cracks. The test results revealed that for a given load position, the girders that were subjected to fatigue loading experienced the same failure mechanism as that for the girders that were only statically loaded [30]. Burns concluded that very little bond strength deterioration was caused by fatigue loading [31].

The effect of high concrete compressive strengths on the transfer and development lengths of prestressing strands was studied by Mitchell et al. [37]. They tested 22 PC beam specimens that had 28-day concrete compressive strengths from 4500 to 12,900 psi and nominal strand diameters of 3/8, 1/2, and 0.6 in. Their test results showed that higher concrete strength produces shorter strand transfer and development lengths. These researchers presented strand transfer and development length equations, based on the ACI expression for strand development length, to account for the effect of concrete compressive strength. Their equations apply to detensioning procedures that involve a gradual release of the prestressing force.

Cousins et al. [15] investigated the strand spacing and concrete cover amounts for specimens that were prestressed with 1/2-in. diameter, grit-impregnated, epoxy-coated, 270-ksi, low-relaxation strands. Ten single-strand specimens, 21 specimens with three strands in one row, and 9 specimens with two rows of three strands were cast. The strand spacing ranged between 2 and 3 in. in 1/4-in. increments, and the concrete edge distance was either 1.5, 1.625, or 1.75 in. for the single-strand specimens and either 1.75, 2.0, or 2.25 in. for the multiple-strand specimens. Nine companion specimens that were prestressed with 1/2-in. diameter, uncoated strands were also cast. The concrete compressive strength when the strands were flame cut was between 4200 and 5200 psi and the average effective strand stress immediately after transfer was between 182 and 191 ksi. Based on the concrete cracking that occurred in some of their single and three-strand specimens, Cousins et al. [15] concluded that the AASHTO Specification [1] requirements for concrete cover and strand spacing are inadequate to prevent concrete cracking when 1/2-in. diameter, grit-impregnated, epoxy-coated strands are used to prestress a concrete member that does not contain any confinement reinforcement around the strands along the strand transfer length. They also noted that the implied AASHTO Specification strand transfer length is conservative for epoxy-coated strands, when the concrete does not crack during prestressing.

Elevated temperatures will affect the bond strength of coated prestressing strands. LeClaire and Shaikh [34] determined that the bond strength begins to diminish at a temperature of 125° F. When the temperature of the epoxy coating exceeds 160° F the bond strength rapidly decreases. At a temperature of 200° F and higher, the bond strength is essentially nonexistent. The loss of bond occurs between the strand and epoxy coating rather than between epoxy coating and concrete. LeClaire and Shaikh have recommended that a 160° F temperature limit be set for detensioning of epoxy-coated prestressing strands. Concern for the potential loss of bond strength due to elevated temperatures of coated strands has prompted the Precast/Prestressed Concrete Institute (PCI) to issue the following statements regarding coated prestressing strands in Section 1.3.4.2 of their Design Handbook [42]: "The behavior of epoxy-coated strands at elevated temperatures is of concern due to softening of the epoxy. Pull-out tests show that there is a progressive reduction in bond strength initiating at about 120° F with virtually complete loss of bond occurring at about 200° F. This behavior necessitates a careful monitoring of concrete temperature at transfer of prestress. Because of the uncertainties in properties noted above, particularly the behavior under elevated temperatures, it is recommended that epoxy-coated strand not be used for pretensioned, prestressed concrete products."

The PCI Ad Hoc Committee on Epoxy-Coated Strand published two articles [40, 41] which addressed the use of coated prestressing strands. Information on material properties; design considerations; temperature limit; and handling, installing and stressing of coated strand are provided in the guidelines. For strand transfer and development lengths, the recommended requirements [40] are similar to those for uncoated strand in the ACI Building Code [3] and AASHTO Specification [1].

To the authors' knowledge, the results of research studies that investigate strand transfer and development lengths and performance of coated, prestressing strands in thin, PC bridge subdeck panels used in composite slab construction have not been published to date (December 1994). However, as stated previously, Lane [33] has reported that an FHWA research study is currently being conducted on epoxy-coated and uncoated prestressing strands in subdeck panels.

1.3. Questionnaires on Epoxy-Coated Reinforcement

1.3.1. Design Agency Questionnaire

In August 1993, a questionnaire was distributed to bridge engineers in the 50 state departments of transportation; 3 branches of the U.S. Forest Service; 9 Canadian provinces, Northwest Territories, and Puerto Rico transportation agencies; New Jersey Turnpike, New York State Bridge, and New York State Thruway Authorities; and the Port Authority of New York and New Jersey. This survey addressed topics related to the general background of the design agency, types of epoxy-coated reinforcement, epoxy coatings, range of application for coated reinforcement, designs with epoxy-coated prestressing strands, experience with epoxy-coated strand, and epoxy-coated reinforcement details and specifications. The complete results for this survey are given in the Appendix A (Note that all appendixes are found in Volume 2 of this report.)

Sixty out of 67 questionnaires that were sent to the design agencies were returned. Fifty-three (90%) of the agencies which returned the survey stated that they allow or have allowed the use of epoxy-coated reinforcement in bridges structures. However, as of September 1993, only 47 agencies are currently specifying epoxy-coated reinforcement. Some of the reasons given by the seven remaining agencies who have never specified any type of epoxy-coated reinforcement included: extreme care is required in handling and placing the reinforcement to prevent damaging the coating; corrosion is not a significant problem, additional concrete cover and a lower water-to-cement ratio seems to provide adequate protection to uncoated reinforcement; epoxy-coated reinforcement is more expensive than regular steel reinforcement; and salt is not used on the bridge decks in our region of the country. Five design agencies, which had previously permitted the use of epoxy-coated reinforcement, now prohibit its use. Some of these design agencies listed inadequate performance or questionable benefits of epoxy-coated reinforcement as the cause of the change in their design philosophy. When the respondents were asked if changes have been made in their specifications for using epoxy-coated reinforcement since December 31, 1989, 18 design agencies responded in the affirmative. Some of the reasons stated by the agency representatives for changing their criteria included: test results challenging the benefits associated with the use of epoxy coatings, AASHTO Specification change in coating thickness requirements, FHWA recommendations, adoption of the Concrete Reinforcing Steel Institute (CRSI)'s Certification Program for fusion bonding, and desire to reduce coating defects.

The survey addressed four types of epoxy-coated reinforcement: epoxy-coated standard deformed reinforcing bars, prestressing bars, prestressing strands, and welded wire fabric. Epoxy-coated, standard deformed, reinforcing bars are or have been used by 52 out of the 53 design agencies which responded that they have used epoxy-coated reinforcement. Of those only 3, 4, and 16 design agencies stated that they have used epoxy-coated prestressing bars, prestressing strands, and WWF, respectively. Regarding the texture of the epoxy coating, 52 and 3 agencies specify a smooth coating for their standard deformed bars and prestressing bars, respectively, while the 4 design agencies that use epoxy-coated prestressing strands always require that the coating have grit impregnated into the surface. When epoxy-coated WWF has been used, all 16 design agencies that use this type of reinforcement specify a smooth-surfaced coating.

Epoxy-coated reinforcement has been used in many different types of structural elements for bridges. As expected, epoxy-coated, standard deformed, reinforcing bars have been used very extensively in a variety of member types. Of the four responding design agencies that have used coated strands in bridge components, only one agency has used coated strands in PC subdeck panels.

The most common type of coated strands are seven-wire, 270-ksi, low-relaxation strands. Three agencies have specified 1/2-in. diameter strands and one agency has used 3/8-in. diameter strands. Three of the four design agencies who specify grit-impregnated, epoxy-coated strands were uncertain as to whether the center strand is coated and if the grit is essentially uniformly distributed around and along the strands. When epoxy-coated, prestressing strands are used in PC panels or slabs, the minimum concrete cover over the strands varies between 1 and 1-3/4 in. Two design agencies use a minimum center-to-center spacing between individual strands of 2 in. and one agency uses a spacing of 6 in.

The use of confinement reinforcement along the prestressing strand development length in PC panels or slabs is required by two design agencies. According to three agencies the development length for coated strands is assumed to be equal to the length established by the AASHTO

Specification [1] for uncoated prestressing strands. One agency responded that they were uncertain as to how to evaluate the development length for coated strands. Three epoxy-coated strand detensioning procedures were noted by the design agencies: acetylene torches, abrasive saw blades, and slow release of hydraulic pressure.

Each design agency was asked to relate any specific problems that they have experienced with either epoxy-coated, prestressing strands or prestressed concrete members reinforced with epoxy-coated, prestressing strands. The agencies were asked to classify those problems on a scale of nonexistent to significant. Another question on the survey asked the respondent to rate the usage of epoxy-coated, prestressing strands considering all aspects of manufacturing and performance of prestressed members that contain coated strands on a scale of excellent to poor. Unfortunately, the four design agencies which have used epoxy-coated strands responded that they could not really comment on these questions since they had not used epoxy-coated strands often enough. Each design agency was given the opportunity to provide additional comments related to the use of epoxy-coated prestressing strands. Some of the paraphrased replies included: the extra cost of epoxy-coated strand will probably prevent significant use of these strands; coated strands have performed successfully since 1985 in a bridge structure; our agency supports the use of epoxy-coated strands in bridge girders; usually sufficient concrete cover should eliminate the need for coated strands even when salt is applied to the bridge decks; installation problems and chuck slippage can occur with epoxy-coated strands; and quality control issues exist since the epoxy coating has been applied to strands which had been corroded.

1.3.2. Precaster Questionnaire

In August 1993, a questionnaire was distributed to 205 precast prestressed concrete producers from the United States and Canada. This survey addressed topics related to the producer's backgrounds, types of epoxy-coated reinforcement, epoxy coatings and uses, designs with epoxy-coated prestressing strands, experiences with epoxy-coated prestressing strands, and epoxy-coated reinforcement details and specifications. The complete results for this survey are given in the Appendix A.

Seventy-six (about 37%) of the 205 questionnaires that were sent to the precast concrete manufacturers who are members of PCI were returned. Fifty-seven or about 75% of those precasters who returned the survey stated that they have produced precast members with epoxy-coated reinforcement, and 41 or about 54% of the respondents have used epoxy-coated reinforcement in bridge structure members. Some of the reasons given by those producers who have never used any type of epoxy-coated reinforcement included: never specified by the design agencies; fire concern versus corrosion in building construction; suppliers of epoxy-coated strand not available in the local area; and the additional expense for epoxy-coated reinforcement. Three companies that had previously produced precast members containing epoxy-coated reinforcement stated that they have stopped producing these types of members since the projects which required epoxy-coated reinforcement were completed. One company discontinued using epoxy-coated, standard deformed bars since they have experienced embrittlement problems with epoxy-coated hardware.

Of the 57 manufacturers which have used epoxy-coated reinforcement in their precast products, 54 companies have used it in standard deformed reinforcing bars, 2 in prestressing bars, 13 in prestressing strands, 20 in welded wire fabric, and 2 in spiral wire. Regarding the use of a smooth-surfaced, epoxy coating, 54 respondents noted that their companies have used it on standard deformed reinforcing bars, 2 on prestressing bars, 1 on prestressing strands, 20 on welded wire fabric, and 2 on spiral wire. Thirteen precast manufacturers have used a grit-impregnated, epoxy coating on prestressing strands. The survey respondents noted that their companies have not used a grit-textured coating on any other type of coated reinforcement.

The precast manufacturers' survey revealed that more than 19 design agencies have used epoxy-coated, standard deformed, reinforcing bars in bridge girders, beams, and single or double-tee sections. Less than 20 design agencies have used coated-reinforcing bars in columns, multistemmed bridge units, full-depth bridge decks, bridge subdeck panels, piles, and hollow core slabs. Epoxy-coated prestressing strands have been used by only three agencies in bridge girders, hollow core slabs, and piles. Two agencies have used coated strands in full-depth bridge deck panels, while only one agency has used epoxy-coated strands in single or double-tee sections. None of the precasters who responded to the survey indicated that they have used epoxy-coated prestressing strands in bridge subdeck panels. Epoxy-coated, welded wire fabric has been used by 16 design agencies in single or double-tee sections and by only one agency in bridge subdeck panels.

Apparently, epoxy-coated, prestressing strands are not a common type of reinforcement for the prestressed concrete industry. Five of the 13 companies that have used this type of strand have produced precast elements containing epoxy-coated strand only since 1991. Florida Wire & Cable Company is the only epoxy-coated, prestressing strand supplier mentioned by the precasters. Although there are several configurations for prestressing strands, the 1/2-in. diameter, 270-ksi, low-relaxation, seven-wire strand is the most common. Only one precaster has used 3/8-in. diameter, epoxy-coated strand. According to the respondents, the grit that is impregnated in the strand surface is uniformly distributed along the length and around the perimeter of a strand.

Questions on the minimum amount of concrete cover over a prestressing strand and minimum center-to-center spacing between individual strands in several types of structural members were included in this survey. Some of these results are given in Table 1.1. The minimum concrete cover has not exceeded 2 in. and the minimum spacing has not been less than 2 inches.

Table 1.1. Epoxy-coated strand cover and spacing

Structural Element	Minimum Concrete Cover Over Strand (in.)	Minimum Strand Center-Line Spacing (in.)
PC girders	1 1/2 to 2	2
PC slabs or panels	1 to 2	2 to 8
PC single or double tees	1 1/2 to 2	2

The questionnaire included two questions concerning the development length of coated strands. Of the 14 producers of PC members that are reinforced with epoxy-coated, prestressing strands, 8 companies do not perform the design of the PC members; 3 companies base the coated-strand development length on the AASHTO, ACI, or PCI Specifications for uncoated strand; 1 company relies on the strand supplier for providing the development length; and 2 companies were uncertain as to how the development length is obtained. Four of the five PC producers that manufacture slabs or panels stated that confinement reinforcement is provided along the epoxy-coated, strand development length.

Experience with epoxy-coated, prestressing strands was addressed by eight questions in the survey. For the strand-detensioning techniques, both a sudden release using acetylene torches or abrasive saw blades and a slow release of hydraulic pressure have been adopted in practice. The sudden release technique is the most common procedure.

Each manufacturer was asked to state any specific problems they have had with epoxy-coated, prestressing strands and concrete members reinforced with these strands. Some manufacturers noted that they have not used epoxy-coated, prestressing strands often enough to provide any comments. For those who relayed specific problems, slippage of a strand at the end chucks, difficulties in removing the chucks from the strand ends after cutting, and difficulty of handling coated strands as opposed to uncoated strands were noted by 7 of the 14 PC producers that have used epoxy-coated, prestressing strands. Even though one of the responding producers mentioned that concrete cracking of members reinforced with coated strands has occurred when members were prestressed, no special precautions are normally taken to minimize concrete cracking. Most of the companies categorized any problems associated with the use of epoxy-coated, prestressing strands as being moderate problems. When the producers were asked to rate the usage of epoxy-coated strands considering all aspects of manufacturing and performance of members, five producers rated epoxy-coated strand usage as fair, one producer chose good, and another producer chose very good.

The precasters were asked to express any additional comments related to the usage of epoxy-coated, prestressing strands. Some of their responses included: we have not seen specifications on epoxy-coated strands; epoxy-coated strands are more difficult to handle; chuck seating requires more strand movement; strand slippage can occur at the chucks when epoxy-coated strands are used; steam curing could be a problem with coated strands since the coating softens at about 150°F; and fire resistance needs to be addressed when epoxy-coated strands are used.

2. DESCRIPTION OF TEST SPECIMENS

2.1. Introduction

A total of 115 test specimens were constructed from 17 concrete castings for the purposes of establishing a recommended minimum thickness for PC subdeck panels containing coated strands and epoxy-coated WWF, measuring the transfer and development lengths of coated strands, and measuring the transfer and development lengths of uncoated strands for comparative purposes. Sixty-seven of the specimens contained coated strands and the remaining 48 specimens contained uncoated strands. All of the specimens were categorized as either T-type or D-type on the basis of the strand behavior being studied. Seventy-five T-type specimens were used to establish the recommended minimum thickness of subdeck panels containing coated strands and also to measure the transfer lengths of both coated and uncoated strands. Forty D-type specimens were used to measure the development lengths of both coated and uncoated strands. Descriptions of the T-type and D-type specimens along with the materials used to construct them are given in this chapter.

Initial testing was performed on two thicknesses of 12-in. wide, T-type specimens containing two coated strands to establish a preliminary, minimum panel thickness. Subsequent testing of the 4, 6, and 36-in. wide, T-type specimens confirmed the preliminary panel thickness, established the transfer lengths of coated and uncoated strands, and indicated the influence of specimen size and multiple strands on the strand transfer lengths. Testing of the 4, 6, and 36-in. wide, D-type specimens was conducted to measure the development lengths of coated and uncoated strands and to observe the influence of specimen size and multiple strands on the strand development length. A summary of the specimen types, number of strands in each specimen, and the strand coating is provided in Table 2.1.

Table 2.1. Specimen quantities by type, strand coating, and number of strands

Strand Coating	T-type Specimens			D-type Specimens	
	1 Strand	2 Strands	6 Strands	1 Strand	6 Strands
Coated	4	36	11	8	8
Uncoated	8	12[1]	4	16	8

[1]Twelve specimens from Cast No. 1 were used to establish testing procedures and verify equipment operations only.

For organizational and record-keeping purposes, a specimen identification system was devised that uniquely describes the pertinent characteristics of each specimen. The notation, as illustrated in Figure 2.1., includes a cast number, specimen thickness in inches, specimen type, strand coating, and specimen number. The cast number, which ranged from 1 to 17, was assigned to a group of specimens belonging to the same concrete casting. The thicknesses of the T-type specimens ranged in 0.5-in. increments from 2.5 to 4.0 in., while the D-type specimens were always 6.0-in. thick. The specimen type designations were T for the transfer length specimens and D for the development length specimens. The strand coating was designated as C for coated strands and U for uncoated strands. The last number in the specimens identification marking was the specimen number. This number also identified the position of each specimen in the prestressing bed. Except for Cast No. 1, the width, length, and strand locations at both ends of all specimens were measured after the specimens were tested. The specimen width, center-to-center spacing of the strands, edge distances from the center of the nearest strand, and distance from the center of each strand to the top and bottom surfaces of the specimen were recorded to the nearest 1/16 in. The specimen length was measured to the nearest inch. If any visible concrete cracks had developed due to prestressing, their

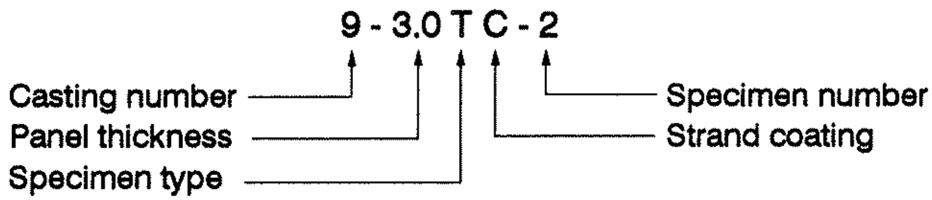
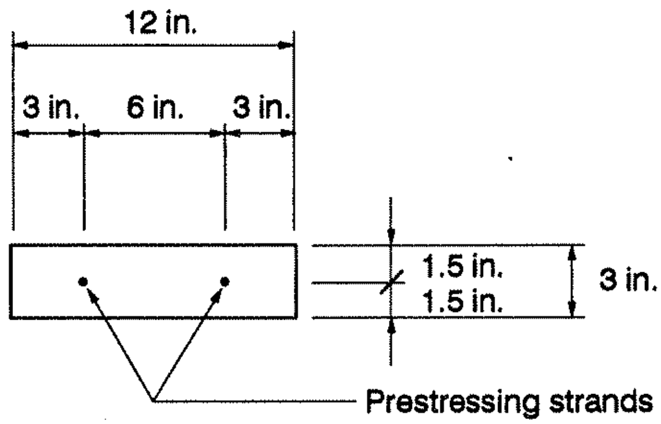
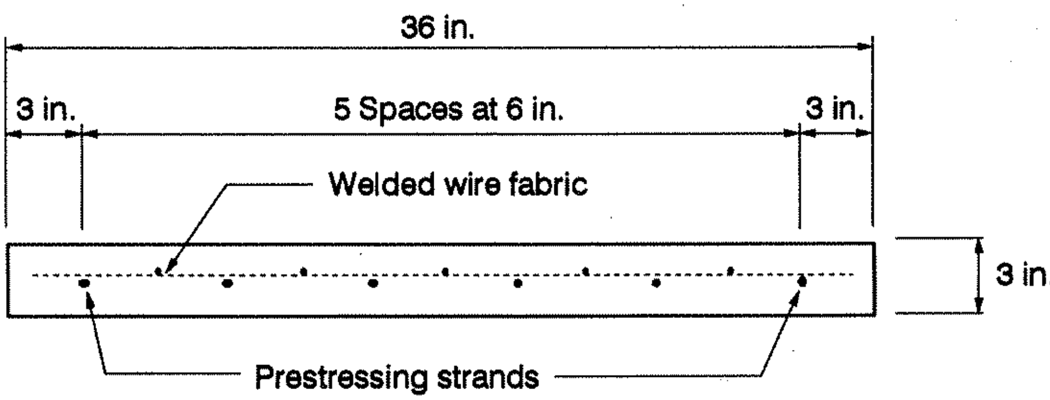


Figure 2.1. Typical specimen identification marking



(a)



(b)

Figure 2.2. Transfer length specimens: (a) 12-in. wide specimen; (b) 36-in. wide specimen

location and length were also documented. A figure of each measured specimen showing the recorded dimensions and any concrete cracks is given in Appendix C.

2.2. Transfer Length Specimens

Table 2.2 lists the characteristics of the T-type specimens that were constructed with each concrete casting. The nominal dimensions for the specimen width, b, thickness, h, and length, L, are listed in the table. Seventy-three of the T-type specimens were cast approximately 6-ft 11-in. long and two of the T-type specimens from Cast No. 13 were about 11-ft 9-in. long. For practical reasons, the thickness for panels reinforced with coated strands was chosen to be in multiples of 1/2 in., but not less than the current standard thickness of 2 1/2 in. used by the Iowa DOT for panels containing uncoated strands. The minimum thickness was 2 1/2 in. and the maximum thickness was 4 in. Each 12-in. wide specimen contained two strands that were horizontally spaced at 6 in. on center about the centroid of the cross section. A nominal 3-in. thick by 12-in. wide specimen is shown in Fig. 2.2(a). Immediately after prestressing the specimens, they were visually inspected for concrete cracks.

After a preliminary thickness was established from the 12-in. wide T-type specimens, testing began on the 36-in. wide specimens. These wider specimens contained six strands spaced at 6 in. on center and located at the midthickness of the panel. The strand positions matched the uncoated strand locations that are in deck panels presently being used by the Iowa DOT. A number of these wider specimens were cast with 6 x 6 - D6 x D6 WWF and were given a raked top surface to simulate the surface condition for an actual deck panel. A cross section for a nominal 36-in. wide, T-type specimen containing six strands and WWF reinforcement is shown in Fig. 2.2(b). After the strands

Table 2.2. T-type specimen parameters

Cast No.	Strand Coating[a]	Nominal Size h (in.)	b (in.)	L (in.)	No. of Speci-mens	Strands per Specimen[b]	WWF[c] and Raked top Surface	Strain Gages	f'_{ci} (psi)	Concrete Age (days)
1	U	2.5	12	84	3	2	No	No	4120	4.0
	U	3.0	12	84	3	2	No	No	4120	4.0
	U	3.5	12	84	3	2	No	No	4120	4.0
	U	4.0	12	84	3	2	No	No	4120	4.0
2	C	2.5	12	83	6	2	No	No	4780	3.0
	C	3.0	12	83	6	2	No	No	4780	3.0
3	C	2.5	12	83	6	2	No	No	4710	2.0
	C	3.0	12	83	6	2	No	No	4710	2.0
4	C	3.0	12	83	12	2	No	No	7240[e]	2.0
5	C	2.5	36	83	4	6	No	No	4640	2.0
6	U	3.0	36	82	1	6	No	No	2910[e]	5.0
7	U	3.0	36	82	1	6	No	No	3980	1.5
8	C	2.5	36	82	1	6	Yes	No	4150	2.0
9	C	3.0	36	82	1	6	Yes	No	4670	2.0
10	U	3.5	4	82	4	1	No	Yes	4050	2.5
11	U	3.0	6	82	4	1	No	Yes	4730	1.0
12	C	3.0	36	82	1	6	Yes	Yes	4420	1.0
13[d]	C	3.0	36	82	1	6	Yes	No	4180	1.5
	C	3.0	36	141	2	6	Yes	No	4180	1.5
14	C	3.0	36	82	1	6	Yes	Yes	4240	3.5
15	U	3.0	36	82	1	6	Yes	Yes	4010	1.5
16	U	3.0	36	82	1	6	Yes	Yes	4780	1.0
17	C	3.0	6	82	4	1	No	Yes	4390	2.0

[a]C = Coated strand; U = Uncoated strand
[b]Strands are located at the mid-thickness of the specimens
[c]Coated WWF used with coated strands; Uncoated WWF used with uncoated strands
[d]Strands over-tensioned to approximately 19.0 kips (83 percent of ultimate tensile strength)
[e]Concrete strength outside of band range shown in Fig. 2.4.

were cut, visual inspections were performed to detect if concrete cracks had developed in these wider specimens.

Strand transfer length measurements were performed on some of the 4, 6, and 36-in. wide, T-type specimens by placing embedment strain gages in the specimens at the level of the prestressing strand(s) and monitoring the induced concrete strains over a regular time interval for up to 18 hours after prestressing the specimen. Specific information regarding these embedment gages is discussed in Section 3.3.4. The 4-in. wide specimens were cast 3 1/2-in. thick to approximately match the size of specimens used by other researchers [12], so that comparisons of the test results obtained in this research could be made with those obtained by others. The 4-in. width for the specimens used in this research needed to be 1/2-in. wider than the width used by the other researchers to allow for the installation of embedment strain gages along the side of the specimens. The 6-in. wide specimens represented an elemental width of the 36-in. wide panel. A 3-in. thickness was used for the 6 and 36-in. wide, T-type specimens. The 4 and 6-in. wide specimens contained a single prestressing strand which was geometrically centered in the cross section of the specimen, while the 36-in. wide specimens contained six strands that were positioned as previously discussed. The 36-in. wide specimens also contained a layer of WWF and were given a raked top surface. The test results for these three specimen widths were used to study whether the 6-in. strand spacing and the provided concrete edge cover could affect the strand transfer length.

Except for the specimens in Cast No. 13, each of the strands in the other specimens were pre-tensioned to approximately 17.2 kips. This force produced a stress in a strand equal to about 75% of the ultimate tensile strength of the strand. The specimens in Cast No. 13 were pretensioned to approximately 19.0 kips. This force produced a stress in a strand equal to about 83% of the ultimate tensile strength of the strand. This larger force was applied to produce a larger stress condition along the strand transfer length, so that a qualitative measure could be implied on the concrete crack resistance for 3-in. thick specimens containing coated strands. The strand forces that were present just prior to prestressing the specimens are discussed in Section 5.2.

2.3. Development Length Specimens

Figure 2.3 shows the nominal 4, 6, and 36-in. wide, D-type specimens that were tested to measure the development lengths for the coated and uncoated prestressing strands. The widths, number of strands, and horizontal positioning of the strands used for the development length specimens matched those used for the transfer length specimens. Table 2.3 lists the characteristics for the D-type specimens that were constructed. The 4-in. wide, single-strand specimens were constructed to match the size of specimens used by other researchers [13], and they were also used to investigate the effect of concrete edge cover on the strand development length. The 6-in. wide and the 36-in. wide specimens were studied to establish the effects of strand spacing on the strand development length. All of the D-type specimens were cast 6-in. thick by 11-ft 9-in. long with the centroid of the prestressing strand(s) positioned 2 in. above the bottom of a specimen. In addition, the 4 and 6-in. wide specimens contained a No. 4 reinforcing bar positioned with its centroid 1 7/8-in. below the top surface of the specimen, while the 36-in. wide specimens contained four No. 4 bars at this location. The No. 4 bars were provided to prevent concrete flexural cracking at the top of the D-type specimens when they were lifted from the prestressing bed and positioned into the development length test frame.

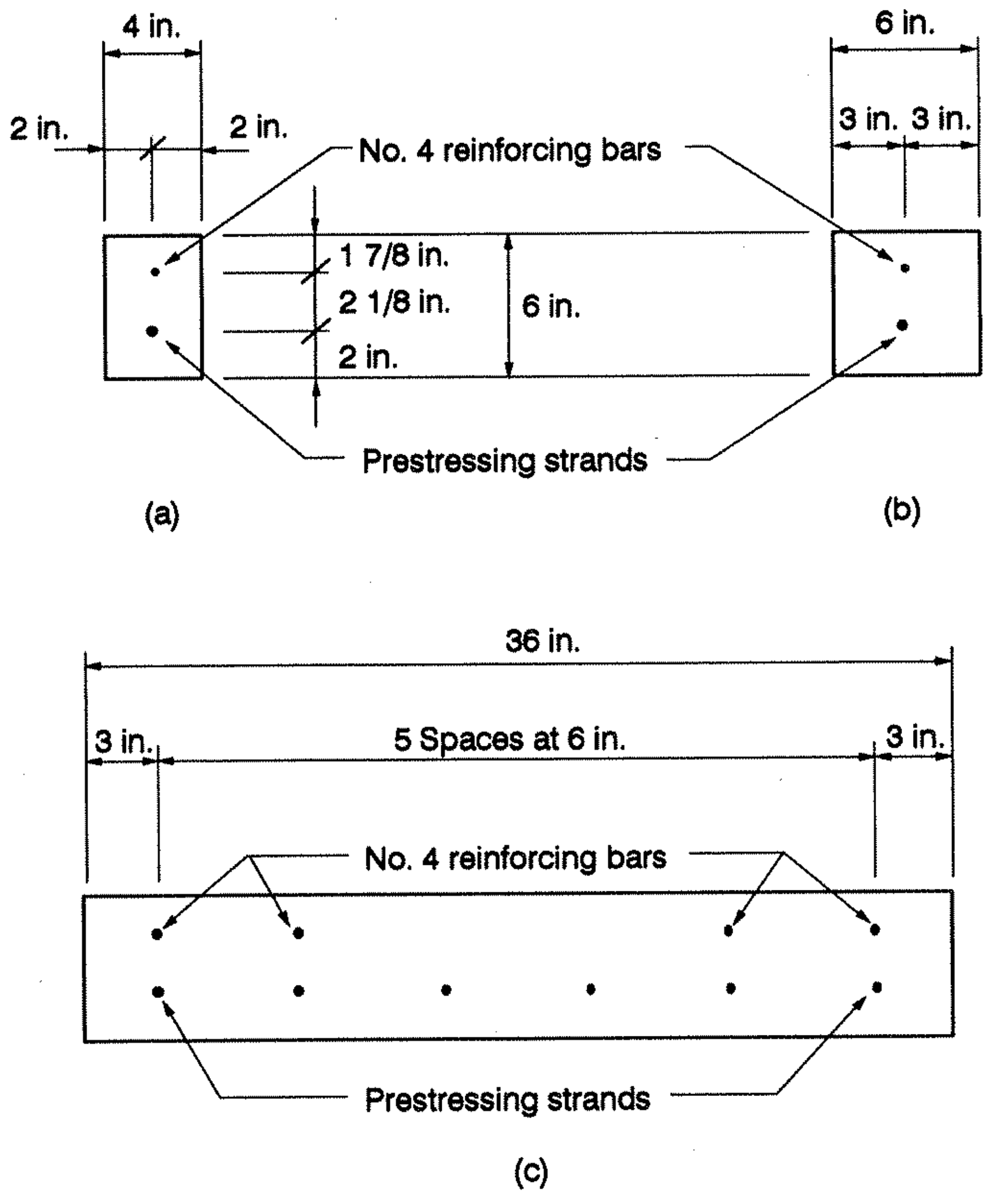


Figure 2.3. Development length specimens: (a) 4-in. wide specimen; (b) 6-in. wide specimen; (c) 36-in. wide specimen

Table 2.3. D-type specimen parameters

Cast No.	Strand Coating[a]	Sizes (in.) h	b	L	No. of Specimens	Strands per Specimens[b]	f'_{cd} (psi)	Concrete Age (days)
6	U	6.0	36	141	2	6	2920[c]	5.5
7	U	6.0	36	141	2	6	4890	5
8	C	6.0	36	141	2	6	5120	12
9	C	6.0	36	141	2	6	5710[d]	4
10	U	6.0	4	141	4	1	5350[d]	5.5
	U	6.0	6	141	4	1	5350[d]	5.5
11	U	6.0	4	141	4	1	6150[d]	2.5
	U	6.0	6	141	4	1	6150[d]	2.5
12	C	6.0	36	141	2	6	6140	2
14	C	6.0	36	141	2	6	5230	7
15	U	6.0	36	141	2	6	5440	4
16	U	6.0	36	141	2	6	5870	2
17	C	6.0	4	141	4	1	5130	4
	C	6.0	6	141	4	1	5130	4

[a]C = Coated strand; U = Uncoated strand
[b]Strands are positioned 2 in. above the bottom of the specimens
[c]Concrete strength outside of band range shown in Fig. 2.4
[d]Average value for test duration

2.4. Material Descriptions

2.4.1. Concrete

The concrete mix design, which was selected for the PC panels cast during the research, produced concrete that was similar to the concrte used for the panels that are manufactured at Iowa Precast Concrete in Iowa Falls, Ia. The target mix quantities per cubic yard of concrete were 705 lb of cement (Portland Cement Type I), 1721 lb of coarse aggregate (Martin-Marietta-Ames limestone chips with a 1/2 in. maximum size), 1046 lb of fine aggregate (Hallets-Ames concrete sand with a 3/8 in. maximum grain size), 275 lb of water (0.39 water-to-cement ratio), an admixture to develop a 6% air entrainment in the wet concrete, and a water-reducer and plasticizer to produce a 4 to 5-in. concrete slump.

Tables 2.4 and 2.5 lists the results of a sieve analysis for the coarse and fine aggregates, respectively. The coarse and fine aggregate gradations satisfied the requirements for Gradation Nos. 6 and 1, respectively, of the Iowa Department of Transportation Standards (Iowa DOT Standards) [21]. The Iowa DOT Standards Section 2407 states that the gradation of the coarse aggregate should meet the requirement listed for Gradation Nos. 3 or 5. However, the maximum aggregate size for Gradation Nos. 3 and 5 is 1 1/2 in. and 1 in., respectively. These maximum aggregate sizes are too large for proper consolidation of the concrete around the prestressing strands in PC panels that are 2 1/2 or 3-in. thick. After discussions with representatives from the Iowa DOT, the Martin-Marietta-Ames 1/2 in. limestone chip coarse aggregate was considered to be an appropriate aggregate for the PC panels.

Two concrete ready-mix producers from Ames, Ia. furnished the concrete for all of the castings. Even though the concrete suppliers exercised care in mixing the concrete, some inconsistencies in workability and strength of the concrete for the different castings were experienced. For some of the concrete castings, water was added to the concrete in the drum on the truck to increase the concrete slump and produce a more workable mix. Except for one concrete casting, the 28-day concrete compressive strength was greater than the required minimum strength. The test results for the concrete strengths are given in Section 5.1.1. The inconsistencies experienced with the concrete were attributed to the small quantities of concrete ordered for each casting.

Table 2.4. Coarse aggregate gradation

Sieve Size	1/2-in.	3/8-in.	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
Percent Passing	100	90	29	4.7	2.0	1.5	1.1	0.8	0.6

Table 2.5. Fine aggregate gradation

Sieve Size	1/2-in.	3/8-in.	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
Percent Passing	100	100	98	86	71	43	8.2	0.5	0.2
Note: Trace amounts of coal and 0.27% material had a specific gravity lighter than 2.0.									

2.4.2. Prestressing Strands

The two types of prestressing strands used for the specimens were a grit-impregnated, epoxy-coated strand ("Flo-Bond") and an uncoated strand. Both strand types were manufactured by Florida Wire and Cable Company of Jacksonville, FL. The bare strand for both products is a 3/8-inch diameter, seven-wire, 270-ksi, low-relaxation, prestressing strand manufactured in accordance with ASTM A-416. The "Flo-Bond" strand has a blue colored epoxy-coating for corrosion resistance, applied in accordance with ASTM A-882/A-882M-91 [6], that surrounds the exterior of the seven-wire bundle. The center wire in the bundle was not epoxy coated. The thickness of the coating varied around the perimeter of the strand. The coating was the thinnest directly on the outer wires of the strand that were wrapped around the center wire and the thickest in the grooves between adjacent wrapping wires. During the curing process for the epoxy, an aluminum oxide grit was impregnated into the coating to create a rough-textured surface to the strand. The grit is needed to produce the bond with the surrounding concrete in a specimen. The density of the grit varied slightly along the length of a strand, but more noticeably around the perimeter of a strand. For the coated strands used, there were two regions of high and two regions of low grit densities which alternated around the circumference of a strand. Tension tests were performed on samples of coated and uncoated strands. The results of these tests are given in Section 5.1.2.

2.4.3. Welded Wire Fabric

Welded wire fabric, identical in size to that specified by the Iowa DOT for deck panels, was used in some of the 36-in. wide T-type specimens to observe if the concrete crack patterns and strand transfer lengths would be influenced by its presence. Four-foot wide by seven-foot long sheets of epoxy-coated and uncoated WWF, each with a 6-in. by 6-in. mesh and 0.06-in.^{2} deformed wires in both the longitudinal and transverse directions (6 x 6 - D6/D6 WWF) conforming to ASTM A497, were cut to size so that the longitudinal wires occurred midway between the strands. The epoxy coating on the fabric did not contain any grit. The fabric was placed directly on top of the strands with the transverse wires in the fabric below the longitudinal wires. The WWF was secured to the strands with wire ties. Epoxy-coated fabric was used in the specimens that contained coated strands, and uncoated fabric was used in the specimens that contained uncoated strands. Whenever a sheet of WWF was used in a 36-in. wide, T-type specimen, a raked surface was also applied to the top surface of the specimen in accordance with Iowa DOT standards. This specimen construction was used to simulate an actual subdeck panel condition. A layer of WWF was not placed in the 4 or 6-in. wide, T-type specimens nor in any of the D-type specimens.

2.4.4. Reinforcing Bars

Longitudinal reinforcing bars were placed in the top of the D-type specimens to resist any accidental flexural stresses that might have been induced by moving these specimens. These bars were No. 4, Grade 40, ASTM A615, deformed bars. These bars were supported by transverse bars which rested on 3 3/8-in. high bar chairs. All of the bars were held in place with wire ties. The reinforcing bars were not epoxy coated.

3. EXPERIMENTAL TESTS

3.1. Introduction

A laboratory test program was undertaken to establish a recommended minimum thickness for PC deck panels containing coated prestressing strands, measure the transfer lengths of coated and uncoated strands, and measure the development lengths of coated and uncoated strands. A total of 17 castings were performed. Ten castings involved coated strands and seven castings involved uncoated strands. Casting No. 1 was performed to establish testing procedures, verify equipment and instrumentation operations, and confirm software program functions. A summary of the laboratory testing program showing the cast number, type of strand coating, and objective of each casting is provided in Table 3.1. The solid circle shown in the table indicates the specific casting objectives for each casting. Detailed descriptions of the testing equipment, instrumentation, and procedures are given in this chapter.

3.2. Test Frames

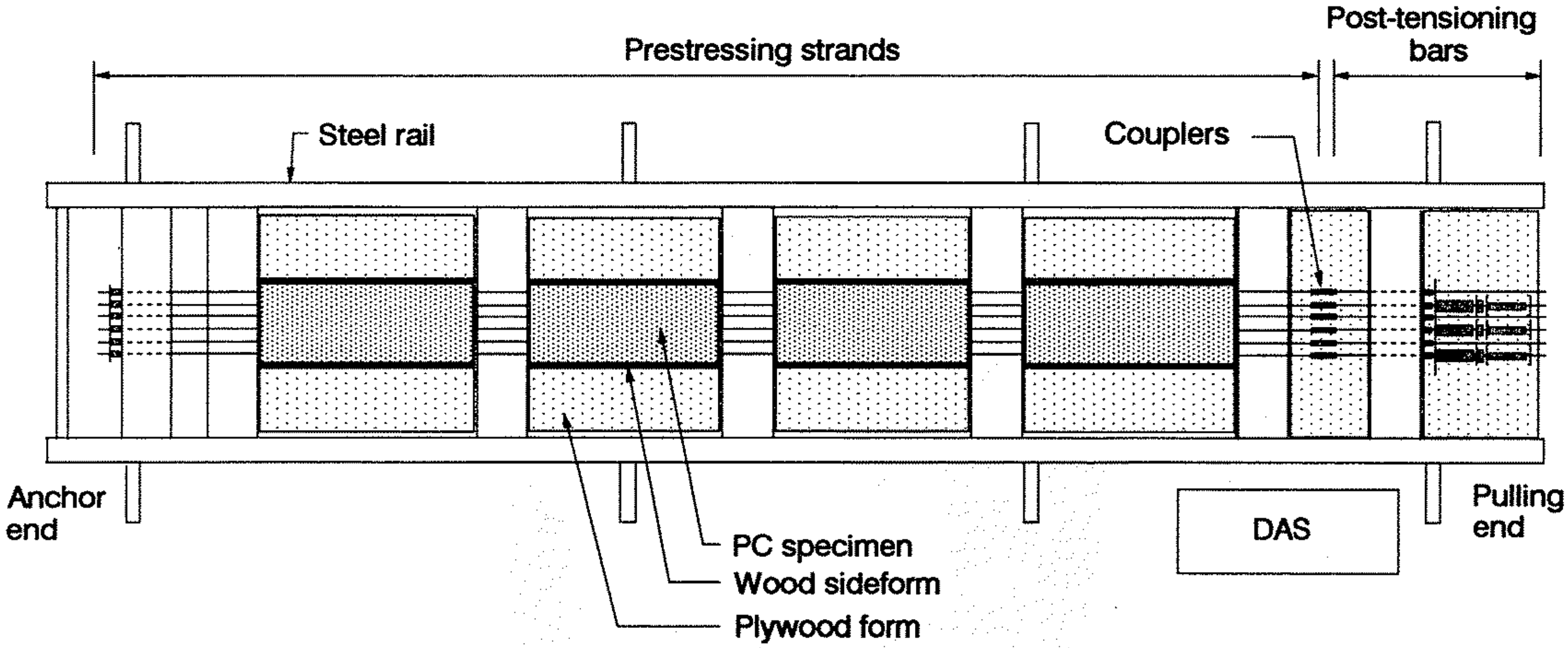
3.2.1. Prestressing Bed

A plan view, longitudinal cross section, and photograph of the prestressing frame containing four 36-in. wide specimens are shown in Fig. 3.1. The self-contained prestressing bed was constructed to cast various widths, thicknesses, lengths, and types of PC specimens. The main elements of the steel frame are two parallel 30-in. deep by 54-ft. long, steel I-shaped rails spaced 9-ft 3-in. on center and 28-in. deep, steel I-shaped headers that span between the rails. Single headers were located at the ends of the specimens and could be positioned at different locations along the length of the frame to accommodate various specimen lengths. Starting at the prestressing end of the

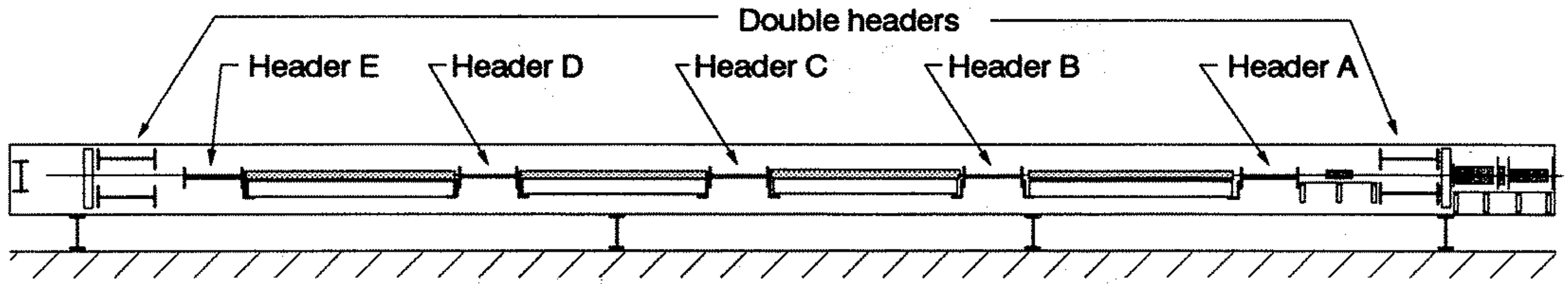
Table 3.1. Casting objectives for the laboratory testing program

Cast No.	Coating Type[1]	Casting Objectives			
		System evaluation	Minimum nominal panel thickness for coated strands	Measure development length	Measure transfer length
1	U	●			
2	C		●		
3	C		●		
4	C		●		
5	C		●		
6	U				●
7	U				●
8	C		●		●
9	C		●		●
10	U			●	●
11	U			●	●
12	C		●	●	●
13	C		●		
14	C		●	●	●
15	U			●	●
16	U			●	●
17	C		●	●	●

[1]C = Coated strand, U = Uncoated strand

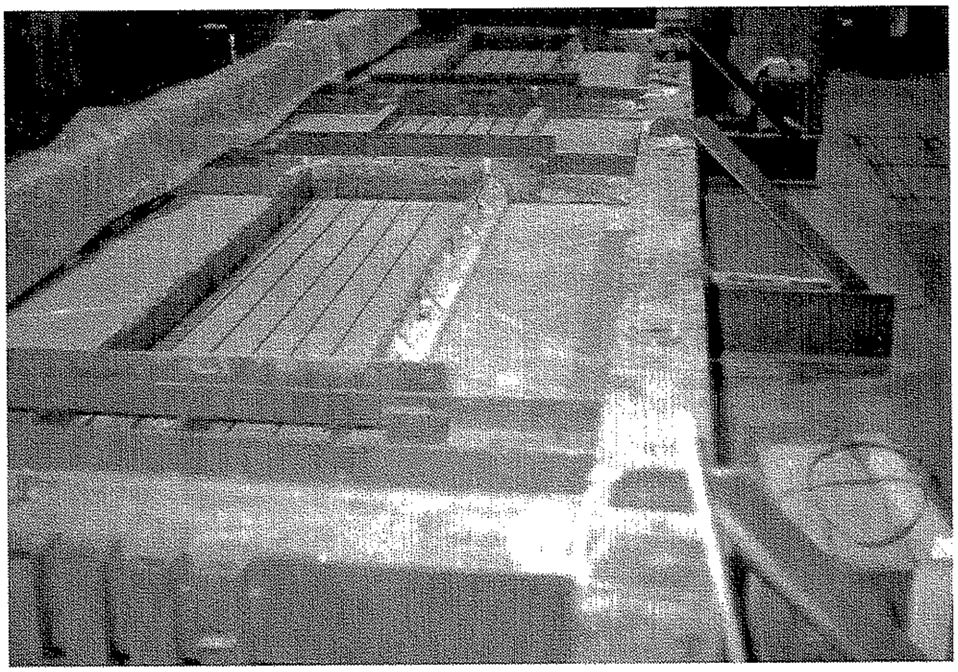


(a)



(b)

Figure 3.1. Prestressing bed: (a) plan view; (b) cross section; (c) photograph



(c)

Figure 3.1. Continued

frame the single headers were alphabetically labeled. Double headers were provided at the frame's designated anchor and pulling ends to resist the prestress force prior to casting the specimens. All of the headers were orientated horizontally with the single headers having their flanges slotted at 6 in. on center to allow the prestressing strands to pass just above their webs. Steel angles were welded to the flanges of the single headers to provide a continuous support shelf for the wood formwork. Detailed layouts showing specimen locations within the prestress bed are provided in Appendix C.

The wood formwork consisted of plywood platforms and wood sideforms which were cut to size from standard dimensional lumber. The 8-ft wide platforms were constructed from 3/4-in. thick plywood and nominal 2-in. by 6-in. joists spaced at 12 in. on center. The plywood was glued with a construction adhesive and screwed to the joists. The platforms provided a smooth continuous casting surface for the bottom of the specimens. Platform height adjustments were made with wood and metal shims placed between the steel shelf angles and the platform framing members to control the position of the prestressing strands relative to the top surface of the platforms. A platform support detail and sideform details for the T-type and D-type specimens are shown in Fig. 3.2.

3.2.2. Development Length Test Frame

Development length tests were performed on the D-type specimens in the rectangular frame shown in Fig. 3.3. The frame was 6-ft 8-in. wide by 11-ft 5-in. long. The base of the frame was constructed with four, 21-in. deep, I-shaped steel beams that supported 24-in. deep, I-shaped steel columns at its corners. An S15x42 reaction beam was erected about 4 ft above the base of the frame so that loads could be applied to a specimen.

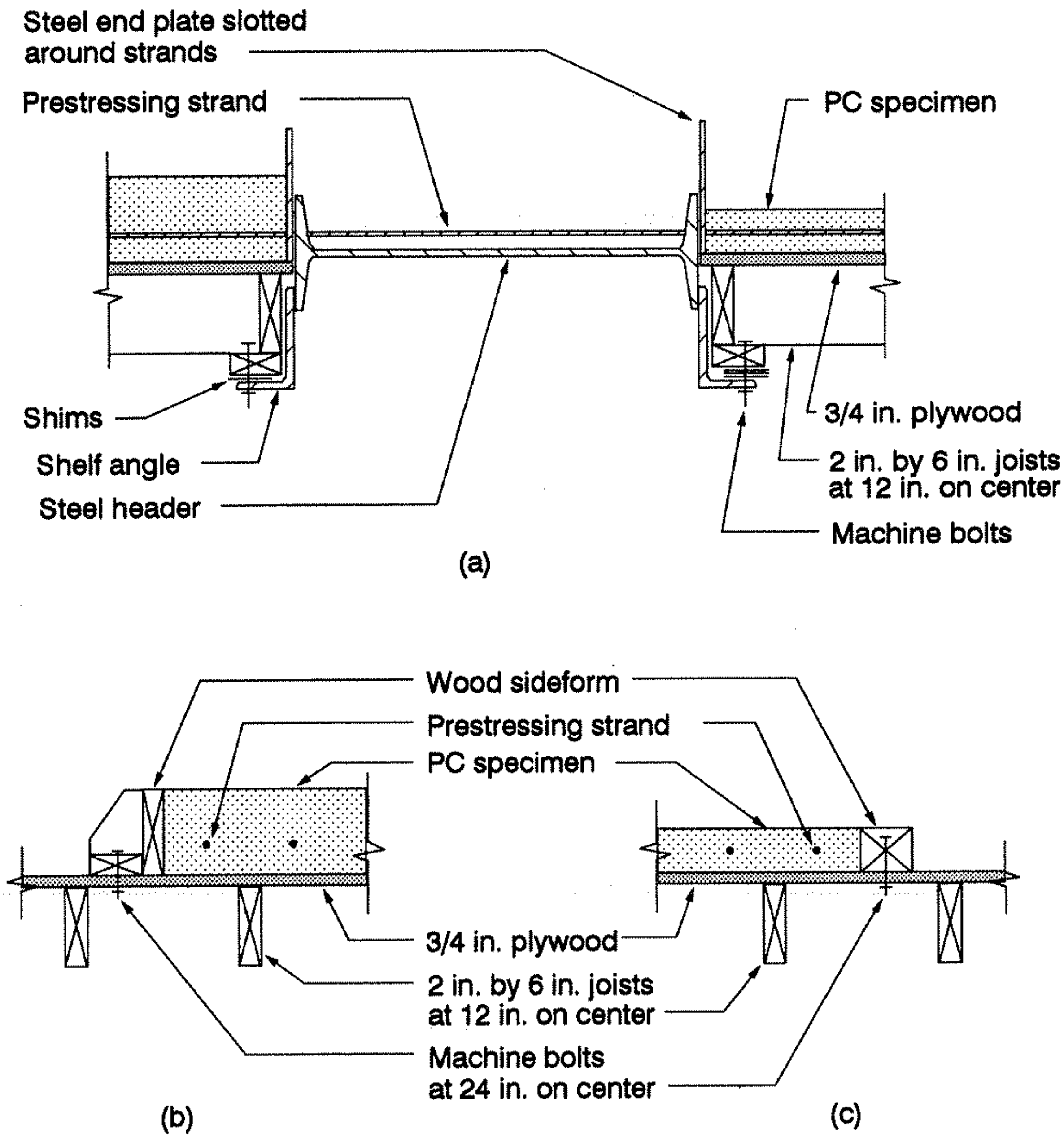


Figure 3.2. Wood formwork details: (a) platform support detail; (b) and (c) sideform details

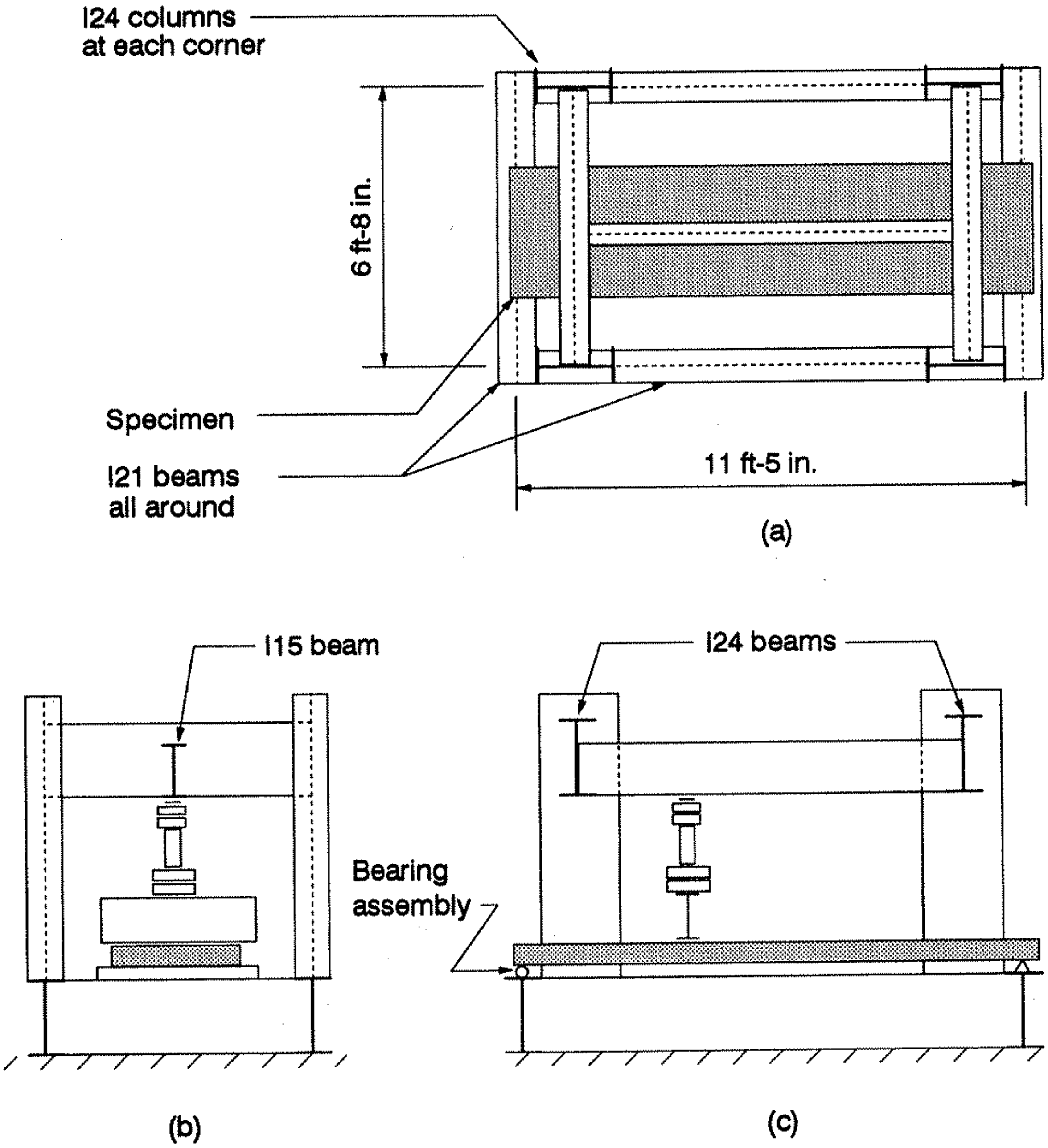


Figure 3.3. Development length test frame: (a) plan view; (b) transverse section; (c) longitudinal section

The ends of a specimen were supported by a bearing assembly on top of the ends of the frame's rectangular base. The bearing assembly consisted of a neoprene pad, a top steel plate, a steel pipe, and a bottom steel plate. The neoprene pad was positioned between the bottom of the specimen and top steel plate and the steel pipe was placed between the two steel plates to permit rotation of the specimen at the supports during testing. The bottom steel plate rested on the top flange of the steel beam at the end of the frame.

Vertical loads were applied to a specimen by pushing with a hydraulic ram against the S15 reaction beam. A spherical loading head distributed the applied force to the S15x42 spreader beam that was placed across the specimen width. A neoprene pad was placed between this spreader beam and the specimen to account for minor irregularities in the top surface of the specimen.

3.3. Instrumentation

3.3.1. Strand Prestress

The forces in the prestressing strands were monitored at regular intervals with electrical resistance strain gages that were adhered to 5/8-in. diameter, high-strength, post-tensioning bars. During the strand tensioning procedure, 50-kip capacity load cells were used to confirm the prestress forces in the strands. Each strand was pulled with a coupling assembly that spliced the strand with a post-tensioning bar. A hydraulic ram reacting against a jacking chair initially applied load to the post-tensioning bar. Since frictional forces were neglected, the axial load in a prestressing strand was equal to the measured force in the post-tensioning bar.

3.3.2. Temperature

Studies [34] have shown that a coated prestressing strand will begin to lose bond with the surrounding concrete at temperatures around 120°F. To measure the highest concrete temperature and to establish the effect of concrete temperature variations on the magnitude of prestressing forces in the strands, the researchers measured the concrete temperature with a thermocouple at regular time intervals during the concrete casting and curing of some of the specimens. The ambient air temperature, the surface temperature of a prestressing strand, and the surface temperature of the steel prestressing frame were also monitored. The two devices that measured the ambient air and frame temperatures were Model No. TJ36-ICSS-14G-12 thermocouples (manufactured by Omega Engineering, Inc. of Standford, Ct.) with a measuring range of 32 to 1400° F (0 to 760° C) and an accuracy of $\pm$ 4.0°F (2.2° C). The two devices that measured the concrete and strand temperatures were Model No. PR-11-3-100-1/4-12-E resistance temperature deflectors (RTDs) (manufactured by Omega Engineering Inc. of New York, N.Y.) with a measuring range of 32 to 1112°F (0 to 600°C) and an accuracy of $\pm$ 1.4° F (0.8° C).

3.3.3. Strand Seating

The strand seating behavior that involved the movement of a strand into a chuck was monitored with stem-type, direct current, displacement transducers (DCDTs) (manufactured by Trans-Tek, Inc. of Ellington, Ct.) which had an accuracy of $\pm$ 0.001 in. Three alternate strands (either Strand Nos. 1, 3, 5 or Nos. 2, 4, 6) were simultaneously monitored. As shown in Figure 3.4, the DCDTs were clamped to a prestressing strand such that the stem of the DCDT pushed against a steel plate at the anchor end of the prestressing frame. An initial 1,000 lb force was applied to a

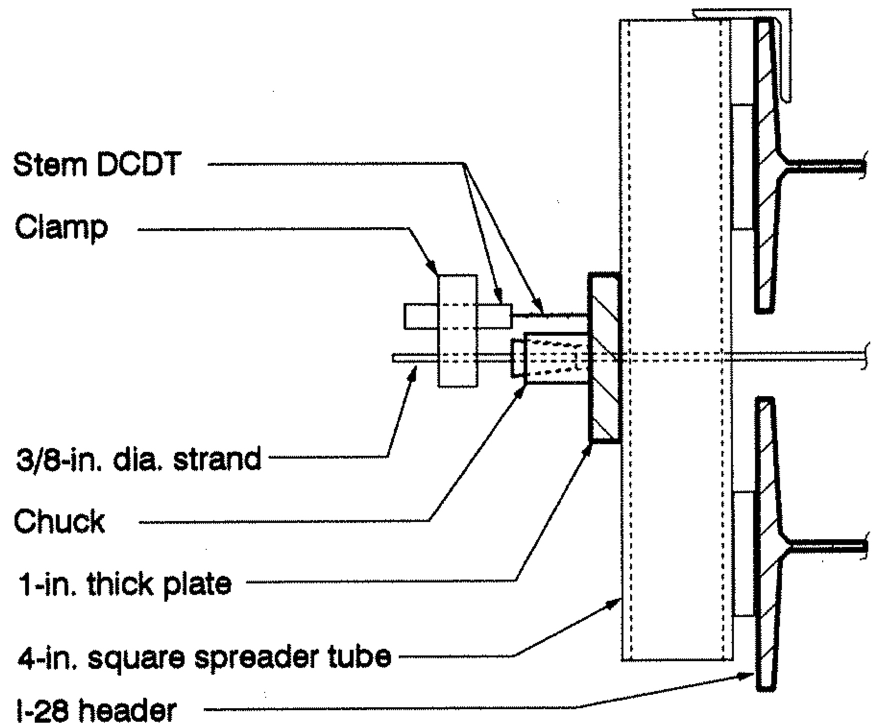


Figure 3.4. DCDT setup for measuring chuck seating at anchor end of prestressing bed

strand to eliminate any slack in the strand and to produce an initial gripping of the strand in the chuck before displacement measurements were recorded.

3.3.4. Strand Transfer Length

The transfer length of a prestressing strand was obtained from the distribution of the concrete strains along the length of a T-type specimen after the prestressing strands were detensioned. To measure the concrete strains in the region of the strand transfer length, the researchers positioned polyester mold PML-30 strain gages (manufactured by Tokyo Sokkikenkyujo, Co. Ltd. of Tokyo, Japan) within some of the specimens prior to casting of the concrete. These gages, which had a 30-mm long gage length, were standard wire gages sealed between two 70-mm long, thin resin plates that had a coarse grit coating to facilitate bonding with concrete. Six PML gages were positioned alternately on both sides and along the length of a monitored strand. The gages were tied between the strand adjacent to a specimen edge and the corresponding specimen side form and between adjacent strands, as shown in Fig. 3.5 for a 36-in. wide specimen. Each gage was orientated at the midthickness of a specimen with the thin dimension of the gage in the horizontal direction. Except for the 4-in. wide, T-type specimens, each gage was located 1.5 in. away from the center of a strand. This horizontal dimension was 1 in. for the 4-in. wide specimens. The longitudinal position of these gages along the specimen length depended on whether a coated or uncoated strand was being monitored. Figure 3.6 shows the locations (g-dimensions) of the embedment strain gages in the 4, 6, and 36-in. wide, T-type specimens. Table 3.2 lists the Specimen No., specimen cross-sectional width b, and thickness h, strand coating, and g-dimensions from the end of a specimen to the center of the gage length. For the 36-in. wide specimens, the gages were located along one edge strand

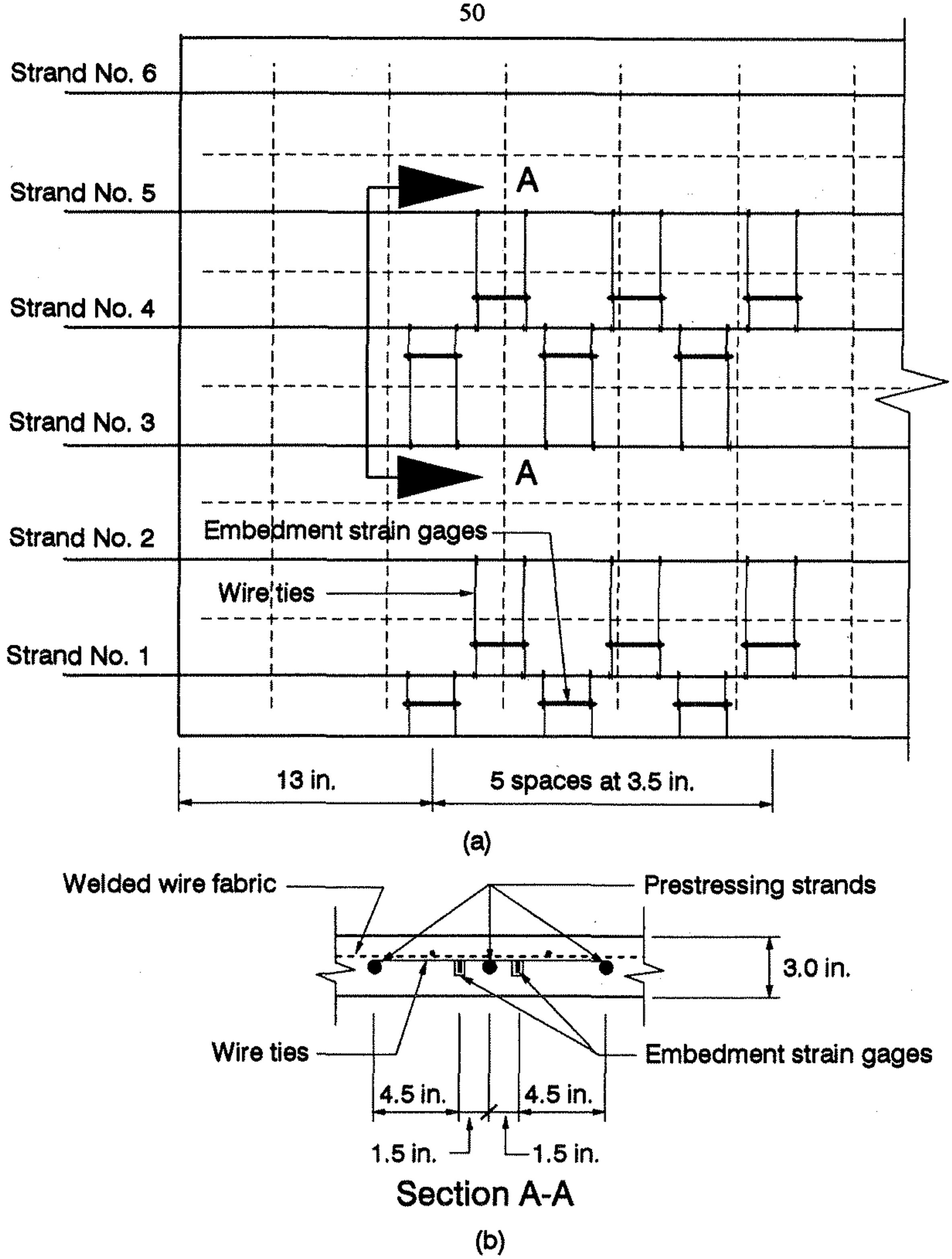


Figure 3.5. Embedment strain gages for transfer length specimens: (a) gage layout; (b) gage mounting detail

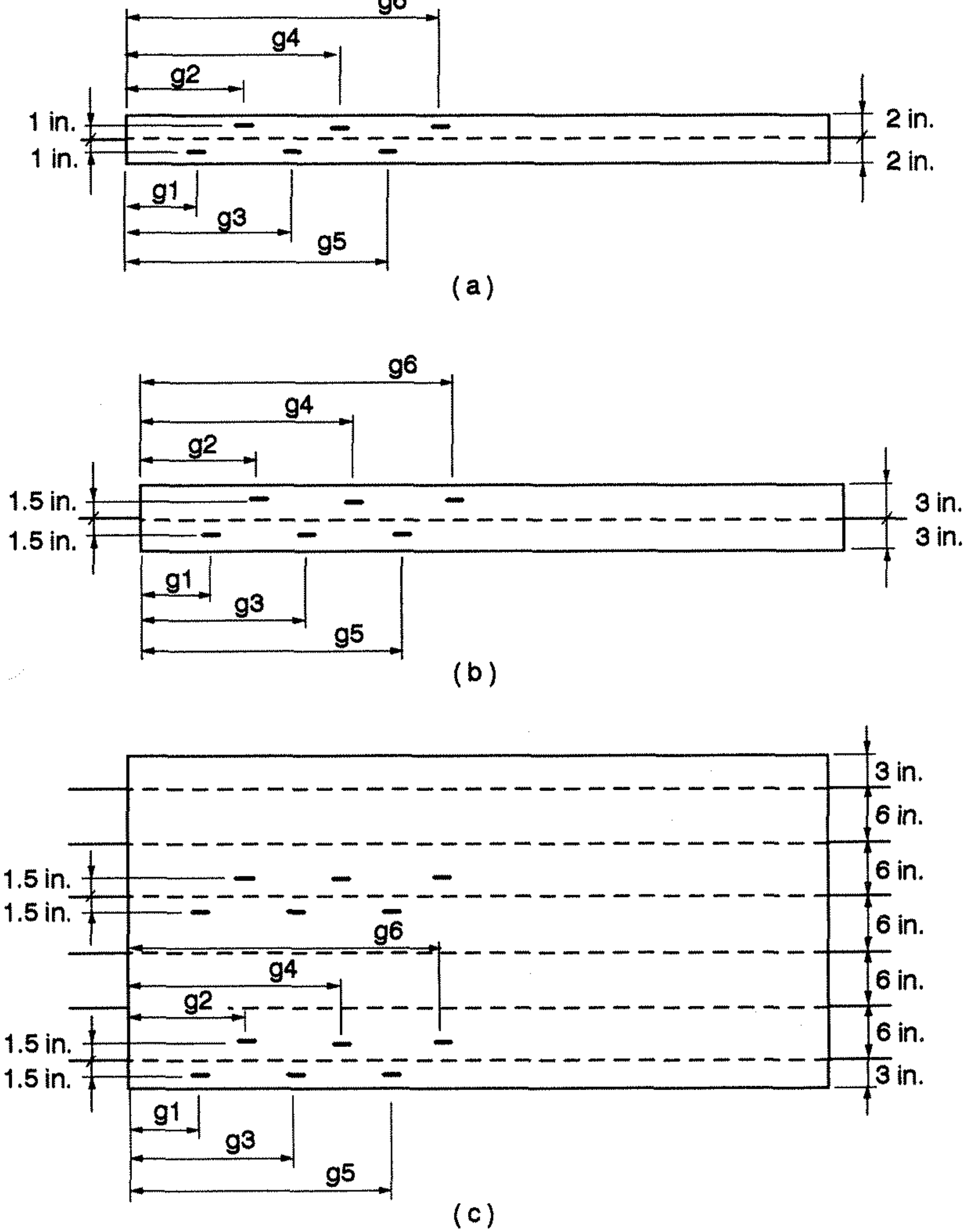


Figure 3.6. Locations of embedment strain gages: (a) 4-in. wide specimen; (b) 6-in. wide specimen; (c) 36-in. wide specimen

Table 3.2. Longitudinal locations of embedment strain gages

Specimen No.	Specimen Size (in.)		Strand Coating[a]	Distance from End to Gage (in.)					
	b	h		g1	g2	g3	g4	g5	g6
10-3.5TU-6 10-3.5TU-7	4	3.5	U	12	19	24	29	34	41
11-3.0TU-6 11-3.0TU-7	6	3.0	U	12	19	24	29	34	41
12-3.0TC-2	36	3.0	C	5	8.5	12	15.5	19	22.5
14-3.0TC-2	36	3.0	C	6	10	14	18	22	26
15-3.0TU-2 16-3.0TU-2	36	3.0	U	13	19	25	31	37	41
17-3.0TC-6 17-3.0TC-7 17-3.0TC-8	6	3.0	C	5	8.5	12	15.5	19	22.5

[a]C = Coated strand; U = Uncoated strand

(Strand No. 1) and one inside strand (Strand No. 4). Seven coated strands and eight uncoated strands were monitored with the embedment strain gages.

3.3.5. Strand Development Length

Two types of displacements were measured during the testing of the development length specimens. Vertical deflections were measured at the bottom surface of a specimen at the locations noted by the solid circles shown in Fig. 3.7. This location corresponded to the transverse line load that was applied at the distance X from the end of the specimen. The deflections were monitored with string-type DCDTs (manufactured by Celesco Transducer Products, Inc. of Canoga Park, Calif.), which had an accuracy of $\pm$ 0.001 in. Small pieces of thin wood blocks were bonded with a 5-minute epoxy glue to the bottom surface of a specimen at the deflection points, so that the string of the DCDT could be attached to a U-shape hook which had been screwed into the wood block. With the base of the DCDT weighted down to the floor of the laboratory, the vertical deflection was measured by the vertical movement of the string.

Potential slippage (strand slip) between a prestressing strand and the surrounding concrete at the ends of the development length specimens was monitored with stem-type DCDTs. As shown in Figs 3.7 and 3.8, DCDTs were attached with clamps to all of the strand extensions at both ends of a specimen. The stem of the DCDT was set against the concrete surface directly above the strand at the end of a specimen.

3.3.6. Data Acquisition System

The readings from the instrumentation were recorded by a Hewlett-Packard (HP) Model 3852A data acquisition system (DAS). A computer was connected to the DAS to control the data monitoring operations with computer programs that were written in the HP BASIC language. The

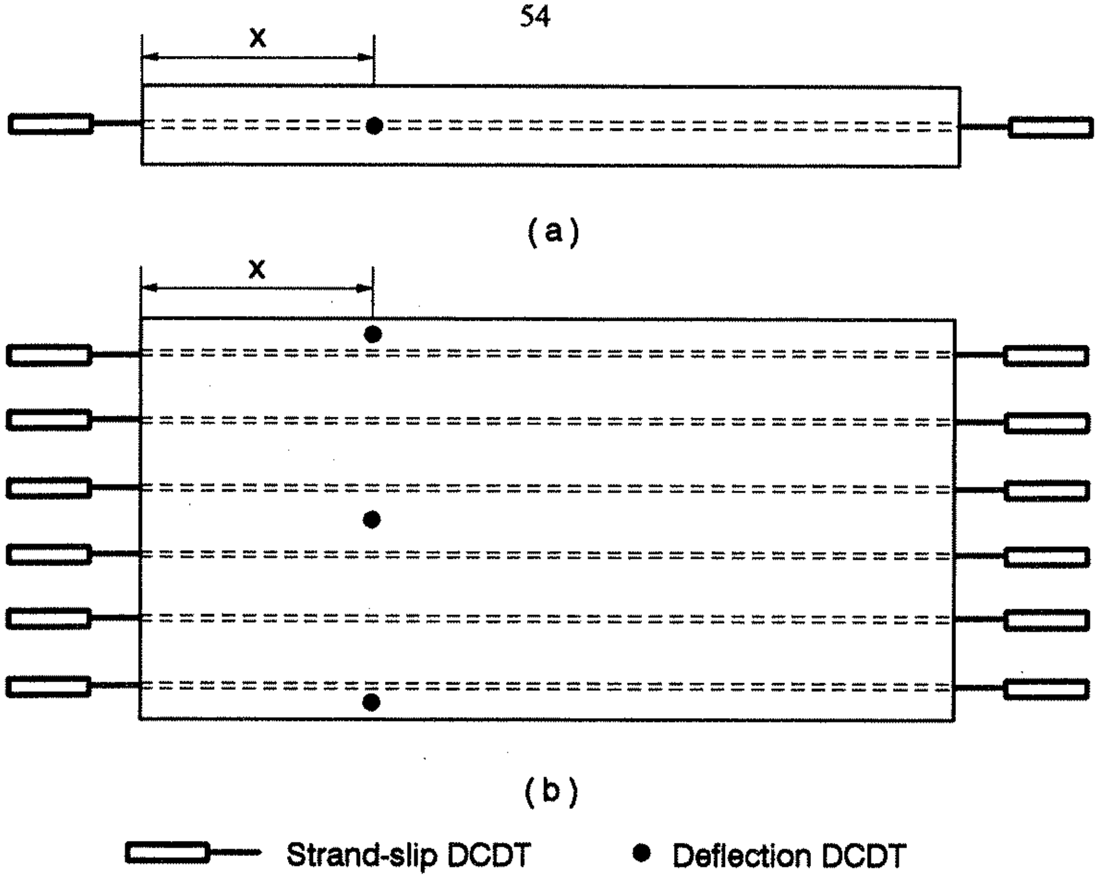


Figure 3.7. Locations of DCDTs for strand development length test: (a) plan view of single-strand specimen; (b) plan view of multiple-strand specimen

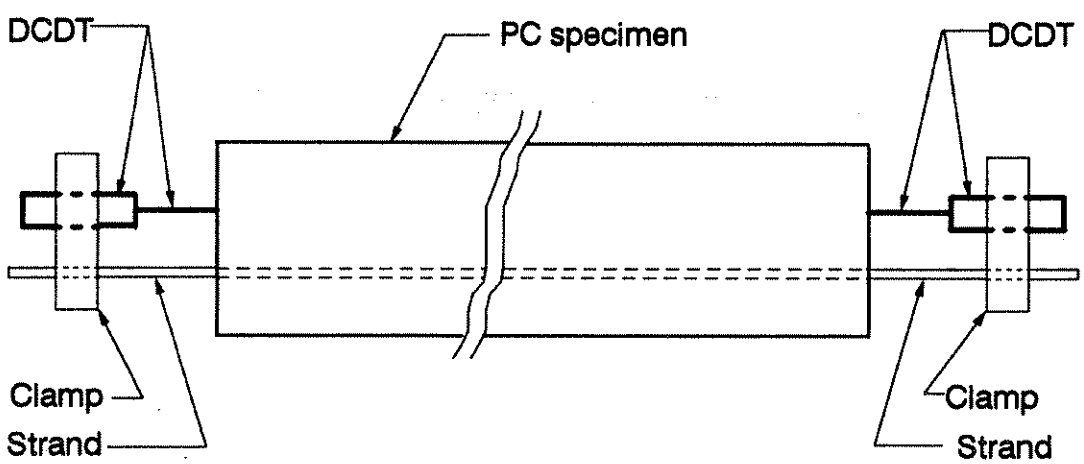


Figure 3.8. DCDT arrangement for strand development length test

programs defined the variable type, channel number, reading increment, data evaluation, and data output destination. Two programs were developed to manage the experimental testing. Program No. 1 continuously monitored the tension force in the post-tensioning bars and when applicable, the temperature readings from the thermocouples, strains from the embedment strain gages, and displacements from the DCDTs for strand movement at the end chucks, during concrete casting and curing, and strand detensioning. Program No. 2 was written to monitor the DCDTs and load cells and to record the strand-slip at the ends of a specimen, deflection of the specimen under the load position, and magnitude of the applied load during the strand development length testing. The program automatically recorded data when a strand-slip occurred and when the peak transverse load was reached.

3.4. Test Procedures

3.4.1. Post-Tensioning Bar Calibration

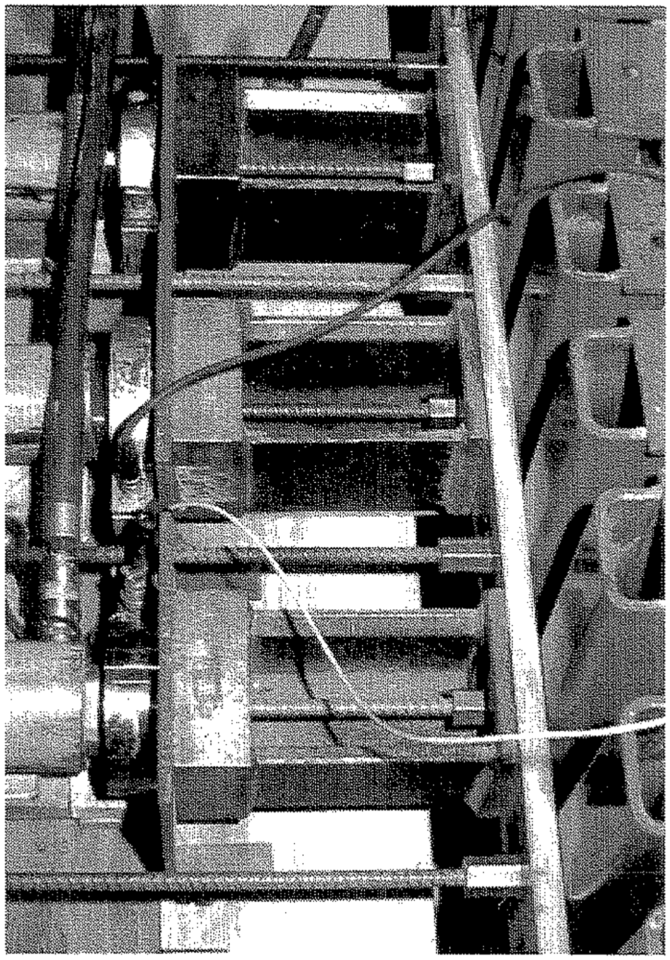
A total of nine 5/8-in. nominal diameter, high strength, post-tensioning bars (manufactured by DYWIDAG Systems International USA, Inc.) were instrumented with electrical resistance strain gages to monitor the forces in the prestressing strands during the strand pretensioning, concrete casting, and specimen prestressing procedures. There were two CEA-06-125UN-120 strain gages (manufactured by Micro Measurements of Raleigh, N.C.) that were mounted on the opposite flat sides of each post-tensioning bar. Each bar was calibrated so that a linear relationship was developed between the measured strain and the corresponding axial load in the bar. With the gages positioned on the opposite sides of a bar, any errors induced by unexpected bending of the bar were minimized by using the average strain reading from the two gages to calculate the axial force in the bar. Before the gages were attached to the bar, the surface in the mounting area was cleaned with sand paper and a conditioner and neutralizer to remove any pits, rust, paint, and oil. The gages were bonded to the bar with an epoxy resin adhesive that was activated by a catalyst. After lead wires were soldered to the tabs of the strain gages, M coat-A, M coat-B and finally M coat-J (manufactured by Micro Measurements) were placed over the strain gages and the surrounding areas to protect the gages from moisture, mechanical, and chemical destruction that would affect the reading from the gages. To minimize potential errors caused by changes in wire resistance associated with different lead wire lengths, each gage was connected with three wire leads in a quarter bridge arrangement in the DAS. To minimize variations in the gage strain readings, each bar was loaded to 22,000 lbs and completely unloaded in a universal testing machine a minimum of five times. After this initial loading sequence, the lead wires for the strain gages were connected to a strain indicator to record the strain in the bar while it was loaded to 22,000 lb in 2000 lb increments. Each bar was loaded and then unloaded three times. The calibration constants for each gage were computed from the recorded strain data by using a linear regression routine. The results of this analysis are given in Table 3.3. The prestressing strand forces were determined with Computer Program No. 1 that periodically measured the strains in the post-tensioning bars and calculated the force on each bar by applying the calibration constants. Since each bar has two strain gages, the program computed two forces and the average force per bar.

3.4.2. Strand Prestressing

Figure 3.9 shows the strand tensioning system at the pulling end of the prestressing bed that was used to tension the prestressing strands to approximately 17.2 kips, which corresponded to 75% of the guaranteed ultimate tensile strength of the strands. The hardware consisted of split-tube-shaped couplers, post-tensioning bars, spreader tubes, jacking chairs, plates, load cells, and hydraulic pumps and rams. The calibrated post-tensioning bars and load cells were used to monitor the strand

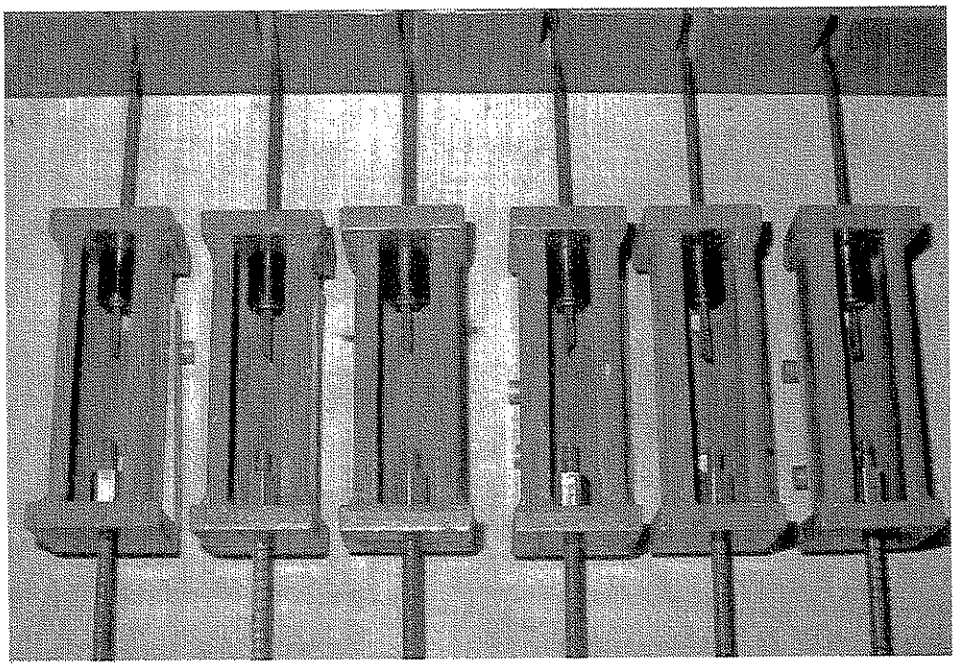
Table 3.3. Post-tensioning bar calibration constants

Bar No.	Gage No.	Calibration Constants		
		Slope (lbs/microstrain)	Intercept (lbs)	Coefficient of Correlation
1	1A	8.1507	-351	1.00000
	1B	8.1126	482	0.99998
2	2A	8.0041	110	0.99996
	2B	8.0314	24	0.99997
3	3A	8.2691	151	0.99995
	3B	8.0978	-115	0.99997
4	4A	8.0867	-259	0.99787
	4B	8.2050	388	0.99994
5	5A	8.0367	-132	0.99999
	5B	8.2704	263	0.99997
6	6A	8.2080	-62	0.99999
	6B	8.1070	193	0.99998
7	7A	8.4459	13	0.99998
	7B	8.4393	-57	0.99999
8	8A	8.1318	-79	0.99996
	8B	8.1586	123	0.99996
9	9A	8.8052	228	0.99992
	9B	8.3094	-180	0.99995

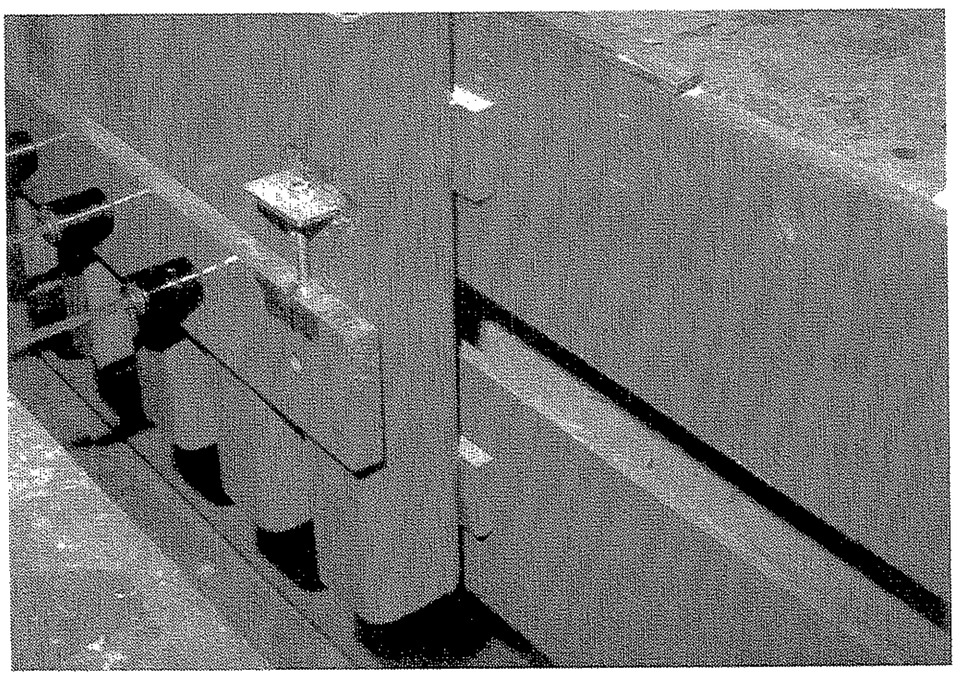


(a)

Figure 3.10. Strand prestressing details: (a) prestressing mechanism; (b) tube-shaped coupler; (c) end anchorage system



(b)



(c)

Figure 3.10. Continued

into position through the plywood platforms. After the strands were tensioned to approximately 75% of their ultimate tensile strength, the heights of the plywood platforms were shimmed to accommodate the correct specimen thicknesses; the slotted end plates were bolted tight making sure that the strands freely pass through the slots; and the positions of the wood sideforms were adjusted relative to the strand locations and bolted tight. When a specimen required a sheet of WWF, the fabric was cut to size, set directly on top of the strands, and secured into position with wire ties. When internal embedment strain gages were used in a T-type specimen, lead wires were soldered to the gages lead wires that were provided by the manufacturer to extend their lengths; the extended lead wires were connected to the DAS; and the embedment gages were secured into position in the specimen with wire ties. One of the final precasting preparation procedures involved applying duct tape over the slots in the steel end plates to prevent the flow of the fresh concrete between the strands and the slots. Just prior to concrete casting, a thin coat of form oil was applied to the wood sideforms and steel end plates to facilitate easier form removal. Immediately before casting of the concrete, any necessary adjustments were made in the strand forces to account for over or under-estimations of the prestress losses.

The concrete casting procedures started with the performance of slump tests in accordance with ASTM C143 [10] on the fresh concrete obtained from a local ready-mix supplier. Adjustments for low slump measurements were made by adding appropriate amounts of water and/or plasticizer to achieve a target slump of 4 to 5 in. Once the target slump was reached, an air entrainment test was performed in accordance with ASTM C231 on the fresh concrete. The target air content was 6%. After placing the concrete into the specimen forms, the concrete was vibrated, screeded, and trowled. During the time that the specimens were being cast, 6-in. diameter by 12-in. tall concrete test cylinders and 6-in. square beam prisms were made in accordance with ASTM C31 [7]. For many of the castings, a thermocouple was inserted into the fresh concrete near the end of a specimen to monitor the temperature changes during the concrete curing period. When a T-type specimen required a roughened finish, the raking of the top surface of the concrete was done after the concrete had started to gain some initial set.

The concrete curing involved covering the specimens with a large plastic sheet for a minimum of 24 hours after which, the plastic cover and wood side forms were removed, and the specimens were left to air dry until the average compressive stress of three concrete cylinders neared 4000 psi. The concrete cylinders and beam prisms were cured in a similar fashion. Concrete cylinders were periodically tested in accordance with ASTM C39 [8] to monitor the curing progress. To aid in possible concrete crack detection, a coat of whitewash paint was applied to many of the specimens.

3.4.4. Strand Release

As shown in Fig. 3.11, the prestressing strands were detensioned by cutting them with an abrasive grinding wheel at the steel headers in the prestress frame when the average compressive strength of three concrete cylinders reached a minimum of 4000 psi. A variety of cutting sequences were utilized to minimize the eccentric compressive loading on the specimens. An example cutting sequence for a casting involving six strands, five headers, and 36-in. wide specimens would begin with Strand No. 3. Cuts would be made in sequence at Headers C, D, B, E, and then A to completely release Strand No. 3 from the prestressing frame. The same cutting pattern at the headers would be repeated in the remaining strands in the order of Strand Nos. 4, 2, 5, 1, and then 6, again cutting each strand completely before proceeding to the next strand. Before each cut on a strand was made, the forces in the post-tensioning bars at the pulling end of the prestressing frame were recorded.

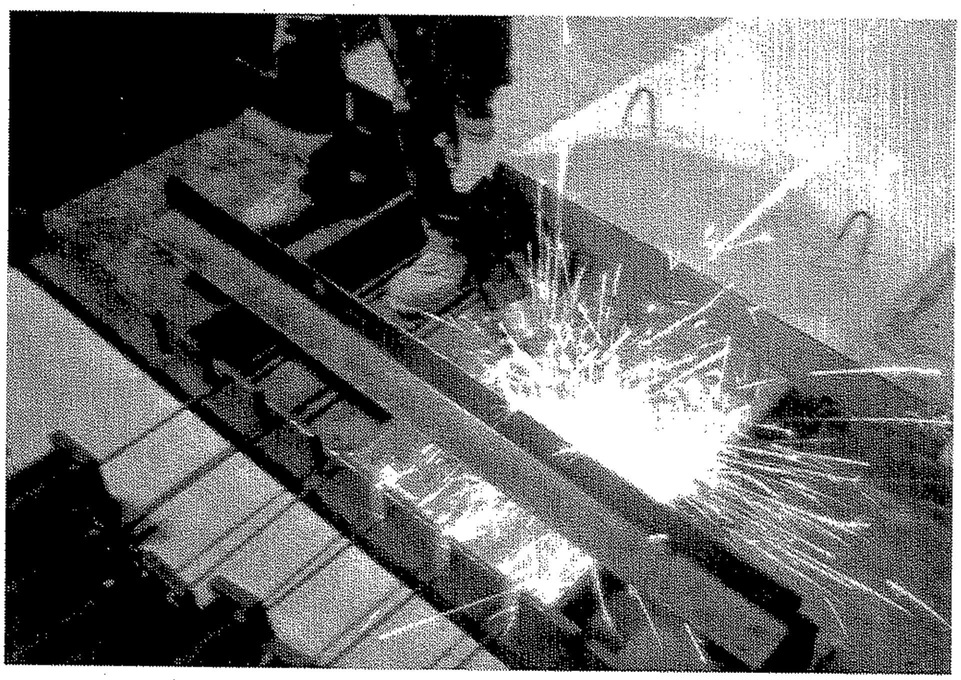


Figure 3.11. Strand-cutting technique

3.4.5. Strand Transfer Length Tests

Embedment strain gages were used to monitor the concrete axial strains for establishing the strand transfer length in some of the T-type specimens. These gages were initialized just prior to cutting the prestressing strands. Concrete strains were measured immediately after each cut was made on a strand during the strand release procedure. After a gaged T-type specimen had been prestressed, concrete strain measurements were taken at one hour increments for up to 18 hours after the last strand had been released.

3.4.6. Strand Development Length Tests

The procedures for the prestressing strand, development length tests included several tasks. After a specimen was lifted and centered in the test frame, the stem-type DCDTs were mounted on the ends of the strands; and the string-type DCDTs were attached to the bottom of the specimen under the load point. Then, the spreader beam, spherical loading head, hydraulic ram, load cell, and filler plates were positioned. Figure 3.12 shows a 36-in. wide test specimen in the test frame. After verifying the connections and operation of all of the instrumentation, transverse loads were applied with a hydraulic pump. When concrete cracks formed in a specimen during the testing, they were marked and numbered on both sides of the specimen. At the completion of a test, the failure mode for the specimen was determined by observing the concrete crack patterns that had developed and by noting whether strand-slip had occurred.

Tests to measure the development lengths of coated and uncoated strands were performed on the 4 and 6-in. wide, single-strand and 36-in. wide, multiple-strand, D-type specimens when the average compressive strength of three concrete cylinders reached a minimum of 5000 psi. To experimentally establish the strand development length, the researchers subjected several essentially

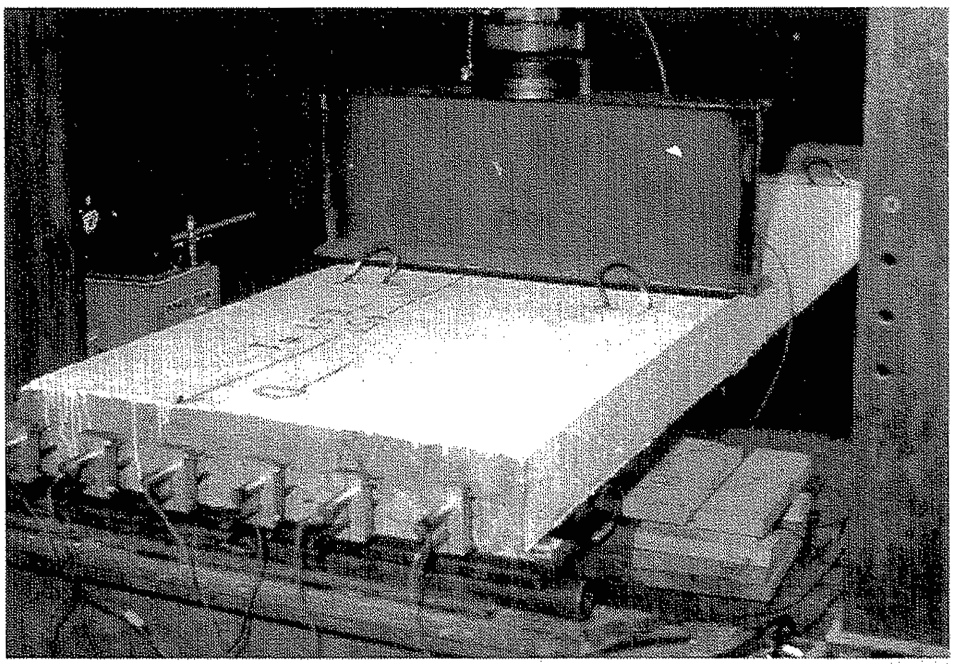


Figure 3.12. Strand development length test arrangement

identical, simply-supported specimens to a transverse load that was placed across their width. For the single-strand specimens, a load increment of 300 lb was used until the first concrete crack appeared. For the 36-in. wide specimens, load increments of 2000 and 1000 lbs were used for loads less than or equal to 10,000 lbs and greater than 10,000 lbs, respectively, until the first visible concrete crack occurred. After the first visual crack was detected in a test specimen, there was no set load increment. If a specimen experienced a flexural failure without the existance of any slippage (end-slip) at the end of a prestressing strand, the strand was fully developed. For the next test at the opposite end of the same specimen or on a new specimen, the load would be positioned closer to the support for the specimen. If the failure of the first end of a specimen was not too severe, the next test could be performed on the opposite end of the same specimen. If the specimen experienced a bond failure as evidenced by a strand-slip before the nominal moment strength of the specimen was reached, the strand that experienced the end-slip was not fully developed. The next test would involve positioning the transverse load further from the end of a specimen. Repeating this procedure on several essentially identical specimens resulted in the convergence of the length X shown in Fig. 3.7 to the strand development length. This approach was used for the single-strand and multiple-strand specimens containing coated or uncoated strands. A strand bond failure was considered to have occurred when the strand-slip measurement reached 0.010 in. Additional information regarding the number and extent of the concrete cracks that were detected on the faces of each specimen is provided in Appendix D.

3.4.7. Strand Tension Tests

To confirm the reported modulus of elasticity values for the prestressing strands that were provided by the manufacturer, the researchers performed strand tension tests in a universal testing machine on three samples each of coated and uncoated strands. Each strand was initially loaded with 1.5 kips to remove the initial curvature in the strand. Then, an extensometer with a 10.0-in. gage length was mounted to the strand. The load was increased to 3.0 kips, and then to 19.0 kips (83% of the guaranteed ultimate tensile strength of the strand) in 2.0-kip increments. For each load increment, strand elongation readings were taken. The ultimate tensile strength for the strands could not be obtained, since special strand gripping devices were not available. The standard chucks for coated and uncoated strands would cause a strand failure within the chuck prior to reaching the ultimate tensile strength of the strand.

4. ANALYTICAL STUDIES

4.1. Bond Mechanisms

When pretressing strands are used as bonded tendons in PC members, the concrete is cast around the strands after the strands are initially prestressed. Once the concrete compressive strength reaches a prescribed value, the strands are cut to induce an initial axial compressive force in the member. After the strands are cut, the prestress force within a particular strand at a location removed from its end anchorage length is resisted by the bond between the strands and the surrounding concrete along their interface within the end-anchorage regions. The tensile stresses in a strand change from zero at the free end of a strand to an effective strand prestress f_{se} at some distance from the end. The distance over which the stress f_{se} is developed is known as the strand transfer length, L_t. When a flexural PC member is subjected to transverse loads, an additional strand embedment length is required for the strand stress to increase to the stress f^*_{su} that corresponds with the nominal moment strength of the member. This additional embedment length is the strand flexural-bond length, L_{fb}. The sum of the strand transfer and flexural-bond lengths is the strand development length, L_d. Figure 4.1 shows a typical strand stress versus strand embedment relationship for a flexural member reinforced with bonded prestressing strands.

For uncoated prestressing strands, the bond mechanism between a strand and the surrounding concrete is a combination of chemical adhesion of the strand to the concrete, friction along the strand to concrete interface, mechanical interlock between the strand and the concrete due to the spiral grooves generated by the outer twisted wires in the strand, and wedging action of the strand known as the Hoyer effect. Chemical adhesion depends on the chemical reactions which take place between the two materials. The friction force is affected by the surface condition of strand and the normal

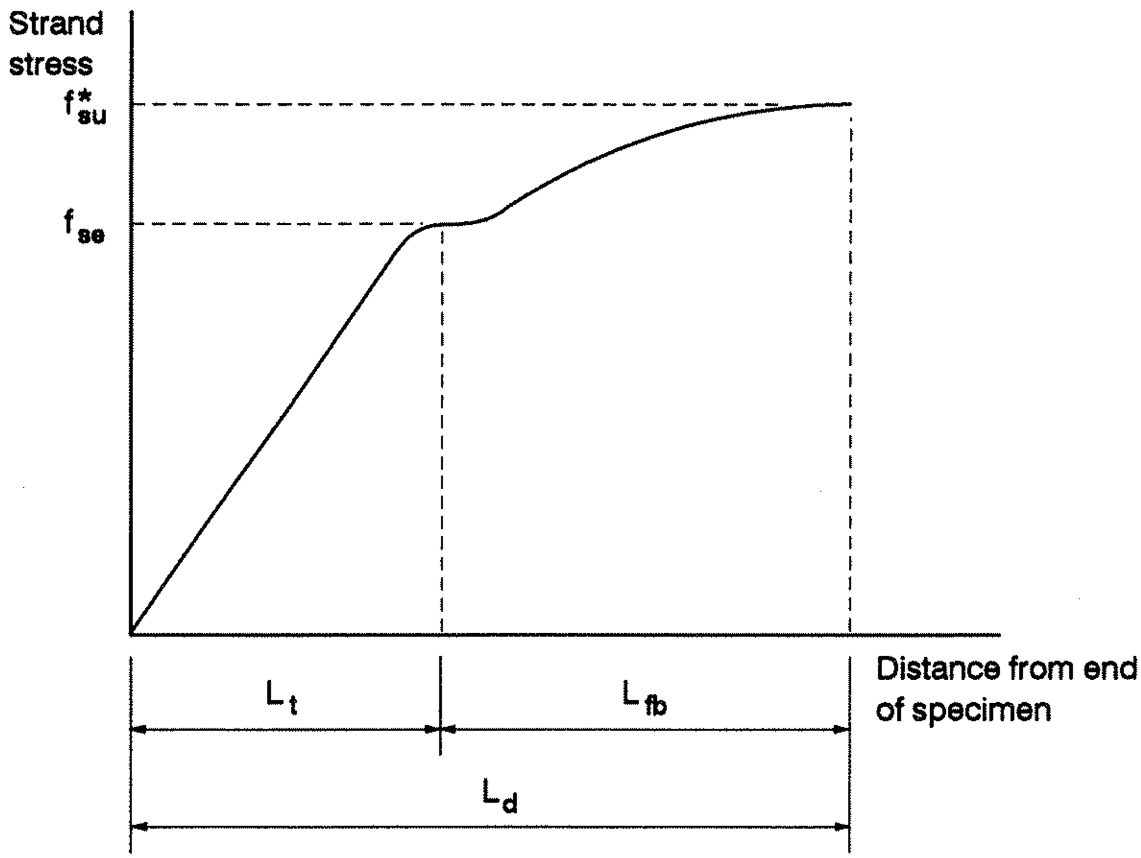


Figure 4.1. Variation of strand stress with distance from end of specimen

force between the strand and the concrete. The Hoyer effect is developed by the change in the diameter of the prestressing strand. When a strand is prestressed, the diameter of the strand is reduced. While the strand remains tensioned, concrete is cast to conform to the geometric shape of the strand. After cutting the strand, the stress at the free end of the strand is zero; therefore, at this location, the diameter of the strand increases; however, a complete rebound to its unstressed diameter is prevented by the confining effects of the surrounding concrete. Within the length of the member, the strand diameter varies since the strand prestress changes along the strand transfer length. An induced normal force between the strand and the concrete, which is caused by the radial expansion of a strand, is a maximum at the free end of the strand and a minimum at the end of the strand transfer length. The reduced strand force within the transfer length corresponds with a shortening of the strand and concrete along this length. The net effect of this behavior produces a wedging action of the strand against the surrounding concrete.

For grit-impregnated, epoxy-coated, prestressing strands, the bond mechanism between the strand and the concrete is different than that for uncoated strands. Dorsten et al. [18] have stated that without the grit impregnated into the surface of the epoxy coating the bond strength between a plain epoxy-coated strand and the concrete is minimal; therefore, external end anchorages would be required to hold a strand with a smooth-epoxy coating. Therefore, the chemical bond, frictional forces at the interface, mechanical interlock from the spiral wire, and wedging action between the epoxy coating on the strand and the surrounding concrete must be also minimal. The epoxy coating seems to act as a barrier between the steel wires and the concrete. The Hoyer effect should be smaller for coated strands than for uncoated strands, since the epoxy coating is compressible. The grit that is embedded in the surface of the epoxy coating must provide chemcial and mechanical anchorage between the epoxy coating and the surrounding concrete. Since the coating is tightly bonded to the steel wires around the perimeter of the strand, the strand is effectively bonded to the surrounding concrete.

4.2. Strand Transfer Length

The strand transfer length is the length of strand embedment in the surrounding concrete that is needed to develop the effective strand prestress. For a pretensioned bonded tendon, the effective strand prestress is the stress remaining in a strand after it has been released from the prestressing bed anchorages and after prestress losses due to elastic shortening ES, creep CR_c, and shrinkage SH, of the concrete; and relaxation CR_s of the strands have occurred. For a low-relaxation prestressing strand with an initial prestress of 75 percent of the ultimate tensile strength f'_s of the strand, the effective prestress f_{se} is given by

$$f_{se} = 0.75 f_s' - (ES + CR_c + SH + CR_s) \tag{4.1}$$

Emperical expressions for calculating each of these losses have been presented by Zia et al. [46]. The AASHTO Specification [1] has adopted some expressions of Zia et al. directly and has modified others. The prestress losses expressions for ES, CR_c, SH, CR_s will be presented for members pretensioned with uncoated, 270-ksi, low-relaxation, prestressing strands. The units of force and length in the expressions associated with the strand transfer length will be pounds and inches, respectively. The concrete elastic shortening loss [AASHTO Specifications Eq. (9-6)] is expressed as

$$ES = \left(\frac{E_p}{E_{ci}} \right) f_{cir} \tag{4.2}$$

where E_p = modulus of elasticity of the prestressing strand, which can be assumed to be equal to 28.5 x 10^6 psi; f_{cir} = compressive stress at the centroid of the tendons immediately after detensioning of the strands and including dead load stresses associated with the self-weight of the member; and E_{ci} = modulus of elasticity of the concrete when the prestress force is applied to the concrete. For normal-weight concrete, E_{ci} can be approximated as

$$E_{ci} = 57{,}000\sqrt{f'_{ci}} \tag{4.3}$$

For members with concentric prestressing, an approximation of the stress f_{cir} is implied in the AASHTO Specification as

$$f_{cir} = \frac{0.69\ f'_s A^*_s}{A_{ce}} \tag{4.4}$$

where A^*_s = cross-sectional area of a prestressing strand and A_{ce} = effective cross-sectional area of the concrete for each prestressing strand. The prestress loss caused by concrete creep [AASHTO Specifications Eq. (9-9)] is expressed as

$$CR = 12\ f_{cir} - 7\ f_{cds} \tag{4.5}$$

where f_{cds} = concrete stress at the centroid of the tendons caused by the superimposed permanent dead loads that are applied to the concrete section. Concrete stresses caused by any dead loads present when the prestress force is applied to the member should not be included in the evaluation of f_{cds}. The concrete shrinkage prestress loss [AASHTO Specification Eq. (9-4)] is given by

$$SH = 17{,}000 - 150\ (RH) \tag{4.6}$$

where RH = mean annual ambient relative humidity expressed in percent. The prestress loss caused by relaxation of the strand [AASHTO Specifications Eq. (9-10A)] is approximated by

$$CR_s = 5{,}000 - 0.10\ (ES) - 0.05\ (SH + CR_c) \tag{4.7}$$

When epoxy-coated, prestressing strands are used, the PCI Ad Hoc Committee on Epoxy-Coated Strand has recommended in their Guide Specification [40] that the losses CR_s for epoxy-coated strands be taken as twice the prestress loss computed from Eq. (4.7). Once all the prestress losses have been determined, the stress f_{se} can be found by applying Eq.(4.1).

For uncoated prestressing strands, the expression for the length L_t in the ACI Building Code Commentary [4] is given by

$$L_t = \left(\frac{f_{se}}{3000} \right) D \tag{4.8}$$

where D = nominal diameter of the prestressing strand. According to Zia and Mostafa [45], the denominator in Eq. (4.8) corresponds to a concrete strength of 3000 psi. This expression is based on the research work performed by Kaar and Hanson [24].

In Article 9.20.2 of the AASHTO Specification [1] that relates to concrete shear strength, the strand transfer length can be assumed to be given by

$$L_t = (50)D \tag{4.9}$$

Another expression for the strand transfer length is implied in the AASHTO Specifications by observing that the strand development length equations in the AASHTO Specifications and ACI Building Code [3] are identical, as discussed in Section 4.4 of this report. Therefore, Eq. (4.8) could be applied for evaluating the strand development length for the AASHTO Specification.

Other expressions have been proposed for the transfer length of uncoated strands. After reviewing and applying a linear regression analysis to the test results of other researchers, Zia and Mostafa [45] noted that the transfer length is more appropriately related to the strand prestress before losses have occurred and to the concrete strength at the time of stress transfer rather than at 28 days. For a sudden release of the strand prestress force, their proposed transfer length expression is

$$L_t = 1.5\left(\frac{f_{si}}{f'_{ci}}\right) D - 4.6 \tag{4.10}$$

where f_{si} = initial stress in prestressing strand before any losses have occurred and f'_{ci} = initial concrete compressive strength when the prestress force is applied to the concrete section. When a gradual release of the prestress force occurs, their proposed expression for the strand transfer length is

$$L_t = 1.3\left(\frac{f_{si}}{f'_{ci}}\right) D - 2.3 \tag{4.11}$$

Immediately after a prestressing strand has been cut, only two types of prestress losses have occurred. These losses are the elastic shortening of the concrete and strand relaxation that has taken place since initial tensioning of the strand. Because the time interval between strand pulling and strand release is normally short (1 to 2 days), the strand relaxation will be minimal; therefore, this prestress loss can be neglected. Applying the remaining prestress loss, the stress f_{se} in an uncoated strand immediately after release can be approximated as

$$f_{se} = 0.75\, f_s' - ES \tag{4.12}$$

Another approach to determine the stress f_{se} is to consider the deformation compatibility between an uncoated prestressing strand and the concrete at a cross section beyond the transfer length. If slippage does not occur between a prestressing strand and the surrounding concrete, the internal static force equilibrium between the tension force in the strands and the compression force in concrete requires that

$$f_{se} = \frac{0.75\, f_s'}{\left[1 + \left(\frac{E_p}{E_{ci}}\right)\left(\frac{A_s^*}{A_{ce}}\right)\right]} \tag{4.13}$$

On the basis of the results from a study of the effect of high-strength concrete on strand embedment lengths by Mitchell et al. [37], the following emperical equation for the strand transfer length of uncoated prestressing strands was suggested when a gradual release of the prestress force occurs:

$$L_t = (330 \times 10^{-6})\, f_{pi}\, D \sqrt{\frac{3000}{f_{ci}'}} \tag{4.14}$$

The concrete strength f'_{ci} ranged between 3000 and 7300 psi for their test specimens. Equation (4.14) was established by the slope-intercept method, which involves drawing a best-fit sloping straight line through strain data points along the strand transfer length and drawing a best-fit horizontal line through the data points beyond the strand transfer length. The end of the transfer length is defined as the point where the two lines intersect.

Cousins et al. [14] have developed an analytical model for the force transfer between a prestressing strand and concrete that divides the strand transfer length into elastic and plastic zones. Their proposed strand transfer length equation was calibrated by correlations with experimental results that were conducted by themselves and other researchers. By selecting different values of a bond stress parameter in the plastic zone, their transfer length expression, which can be used for both uncoated and coated prestressing strands, is given by

$$L_t = \left(\frac{0.5\, U_t' \sqrt{f_{ci}'}}{B} \right) + \left(\frac{f_{se} A_s^*}{\pi\, D\, U_t' \sqrt{f_{ci}'}} \right) \tag{4.15}$$

where the bond modulus B along the elastic portion of the strand transfer length is highly variable. An average value of 300 psi/in. for B was suggested by Cousins et al. [14]. The nondimensionalized bond stress U'_t, along the plastic portion of the strand transfer length, is expressed as

$$U_t' = \frac{U_t}{\sqrt{f_{ci}'}} \tag{4.16}$$

where U_t = plastic bond stress along the plastic zone of the strand transfer length. The following values of U'_t were suggested:

$U'_t = 6.7$ for an uncoated strand

$U'_t = 10.6$ for an epoxy-coated strand with a low-grit density

$U'_t = 16.5$ for an epoxy-coated strand with a medium to high-grit density

In the PCI Guidelines [40] prepared by the PCI Ad Hoc Committee on Epoxy-Coated Strand, the effects of single and multiple strands on the coated-strand transfer length is addressed. For members containing a single coated strand or members containing multiple coated strands with the spaces between the strands and the concrete cover over the strands large enough, so that the effects of multiple strands can be neglected, the transfer length can be obtained from Eq. (4.9). For members containing multiple coated strands with the spaces between the strands or the concrete cover over the strands not large enough to be ignored, a longer transfer length was adopted as

$$L_t = (65)\,D \tag{4.17}$$

4.3. Strand Flexural Bond Length

The strand flexural-bond length is the additional embedment length beyond the transfer length required to obtain the strand stress f^*_{su}. This stress occurs when the nominal moment strength of the PC member is reached. The units of force and length in the expressions associated with the strand flexural-bond length will be pounds and inches, respectively. In ACI Code Commentary [4], the strand flexural-bond length is given by

$$L_{fb} = \left(\frac{f^*_{su} - f_{se}}{1000} \right) D \tag{4.18}$$

The AASHTO Specification does not directly specify the strand flexural-bond length, but the same length as given by Eq. (4.18) is implied.

If the stress f_{se} is not less than one-half of the ultimate tensile strength of the strand, if an appropriate stress versus strain relationship exists for the strands, and if sufficient strand development length is available, the AASHTO Specification [1] (Eq. 9-17) allows for an approximation of the stress f^*_{su}. For members with bonded prestressing strands and without any additional nonprestressed tension reinforcement, the stress f^*_{su} is approximated by

$$f^*_{su} = f'_s \left[1 - \left(\frac{\gamma^*}{\beta_1} \right) \left(\frac{\rho^* f'_s}{f'_{cd}} \right) \right] \tag{4.19}$$

where f_{cd}' = concrete compressive strength when transverse loads are applied to the member, γ^* = factor for type of prestressing steel (γ^* = 0.28 for low-relexation strands), β_1 = Whitney Stress Block factor for concrete strength (β_1 equals as 0.85 for f_{cd}' up to and including 4000 psi and shall be reduced continuously at a rate of 0.05 for each 1000 psi of strength in excess of 4000 psi, but β_1 shall not be taken less than 0.65), and ρ^* = prestressing steel ratio, which is defined as

$$\rho^* = \frac{A^*_s}{b\, d_p} \tag{4.20}$$

Here b = width of the rectangular cross section for the prestressed member and d_p = effective depth from the compression face of the cross section to the centroid of the prestressing steel. In order to have a ductile failure mode consisting of yielding the tension reinforcement, the AASHTO Specification [1] Eq.(9-20) requires that

$$\rho^{*} < 0.36\ \beta_1 \left(\frac{f'_{cd}}{f^{*}_{su}} \right) \tag{4.21}$$

Some researchers have proposed other expressions for the flexural-bond length of uncoated strands. Zia and Mustafa have suggested a 25% increase in the length specified by the ACI Code Commentary [4] requirement. Their expression is

$$L_{fb} = \frac{1.25\ (f^{*}_{su} - f_{se})\ D}{1000} \tag{4.22}$$

A multiplication factor was introduced by Mitchell et al [37] for Eq. (4.18) to account for the effect of high-strength concrete on the strand flexural-bond length. Their equation is expressed as

$$L_{fb} = \left[\frac{(f^{*}_{su} - f_{se})}{1000} \right] D \sqrt{\frac{4500}{f'_{cd}}} \tag{4.23}$$

Cousins et al. [14] suggested the following expression for the strand flexural-bond length, which could be used for both uncoated and grit-impregnated, epoxy-coated, prestressing strands.

$$L_{fb} = (f^*_{su} - f_{se})\left[\frac{A^*_s}{\pi\, D\, U'_d \sqrt{f'_{cd}}}\right] \tag{4.24}$$

where U'_d = nondimensionalized bond stress along the plastic portion of the strand flexural bond length, which is obtained by

$$U'_d = \frac{U_d}{\sqrt{f'_{cd}}} \tag{4.25}$$

where U_d = plastic bond stress along the plastic zone of the strand flexural-bond length. Values for U'_d were suggested as

U'_d = 1.32 for an uncoated strand

U'_d = 4.55 for an epoxy-coated strand with a low-grit density

U'_d = 6.40 for an epoxy-coated strand with a medium to high-grit density

4.4. Strand Development Length

The strand development length is the prestressing strand total embedment length in the concrete required to obtain the stress f^*_{su}. This length, which is equal to the algebraic sum of the transfer and flexural-bond lengths, is given by

$$L_d = L_t + L_{fb} \tag{4.26}$$

If the actual strand embedment length is less than the required strand development length, the nominal moment strength M_n of the PC member can not be obtained. With this condition, the bond strength between the prestressing strands and the surrounding concrete is inadequate to develop the stress f^*_{su} in the strands. Therefore, strand slippage at the end of the member occurs before a flexural failure develops. Conversely, when the actual strand embedment length is larger than the required strand development length, the flexural strength of the PC member will not be limited by the bond resistance between strands and concrete.

By substituting Eq.(4.8) and (4.18) into Eq.(4.26), both the ACI Building Code [3] (Sec. 12.9.1) and AASHTO Specification [1] (Eq. 9-32) expressions for the strand development length are given by

$$L_d = \left[\frac{f^*_{su} - \frac{2}{3} f_{se}}{1000}\right] D \tag{4.27}$$

Similarly, other expressions for the strand development length can be easily established by adding the corresponding transfer and flexural-bond lengths. Zia and Mustafa's [45] equation for the development length of an uncoated prestressing strand can be expressed as

$$L_d = \left[1.5\left(\frac{f_{si}}{f'_{ci}}\right) D - 4.6\right] + \left[\frac{1.25\ (f^*_{su} - f_{se})\ D}{1000}\right] \tag{4.28}$$

Mitchell et al. [37] have suggested the following expression for strand development length,

$$L_d = (330 \times 10^{-6}) f_{se} D \sqrt{\frac{3000}{f'_{ci}}} + \left[\frac{(f^*_{su} - f_{se})}{1000}\right] D \sqrt{\frac{4500}{f'_{cd}}} \tag{4.29}$$

On the basis of their analytical model Cousins et al. [14] obtained their strand development length by

$$L_d = \left(\frac{0.5\, U'_t \sqrt{f'_{ci}}}{B}\right) + \left(\frac{f_{se} A^*_s}{\pi\, D\, U'_t \sqrt{f'_{ci}}}\right) + [f^*_{su} - f_{se}]\left(\frac{A^*_s}{\pi\, D\, U'_d \sqrt{f'_{cd}}}\right) \tag{4.30}$$

In the PCI Guidelines [40] for using coated strands prepared by a PCI Ad Hoc Committee, a single-strand development length expression [Eq. (4.27)] has been specified for both uncoated and coated strands. As previously stated, this equation is the AASHTO Specification Eq. (9-32).

4.5. Nominal Moment Strength

The nominal moment strength M_n of a rectangular PC member at a location beyond the strand development length, can be established from the longitudinal strains that are induced throughout the depth of the cross section. To evaluate the moment M_n by applying strain compatibility, the following assumptions are needed:

- The prestressing strands and any deformed reinforcing bars are fully bonded to the surrounding concrete; therefore, slippage between the steel reinforcement and the concrete does not occur.

- Plane sections before bending remain plane after bending. Therefore, linear flexural strains exist throughout the depth of a cross section.
- The ultimate concrete strain at the extreme fiber of the compression face is equal to 0.003 in./in.
- The nonlinear compressive stresses that occur at the nominal moment strength of the cross section can be approximated by the Whitney Rectangular Stress Block.
- The tensile stress versus strain relationship for the prestressing strands can be represented by the curve presented in the PCI Design Handbook [42].
- The modulus of elasticity of the prestressing strands E_p and the nonprestressed bar reinforcement E_s are equal to 28,500 and 29,000 ksi, respectively.

Figure 4.2 shows a rectangular-shaped, PC member with the prestressing strands located below the centroid of the cross section and the nonprestressed reinforcement positioned near the compression face of the member. In this figure, the dimensions c = distance from the neutral axis to the extreme compression fiber, d_o = depth from the compression face to the centroid of the nonprestressed bar reinforcement, e = eccentricity of the centroid of the prestressing strands that is measured from the centroid of the cross section, and e_o = eccentricity of the centroid of the nonprestressed reinforcement that is measured from the centroid of the cross section.

The prestressing strand tensile strain, which corresponds with the strength M_n of the PC member, can be expressed as the sum of the strand strains associated with three load stages, as discussed by Nawy [38] in Section 4.11.1 of his textbook. The first load stage involves the effective prestressing strand strain ϵ_1 that is induced by the stress f_{se}. This strand strain is given by

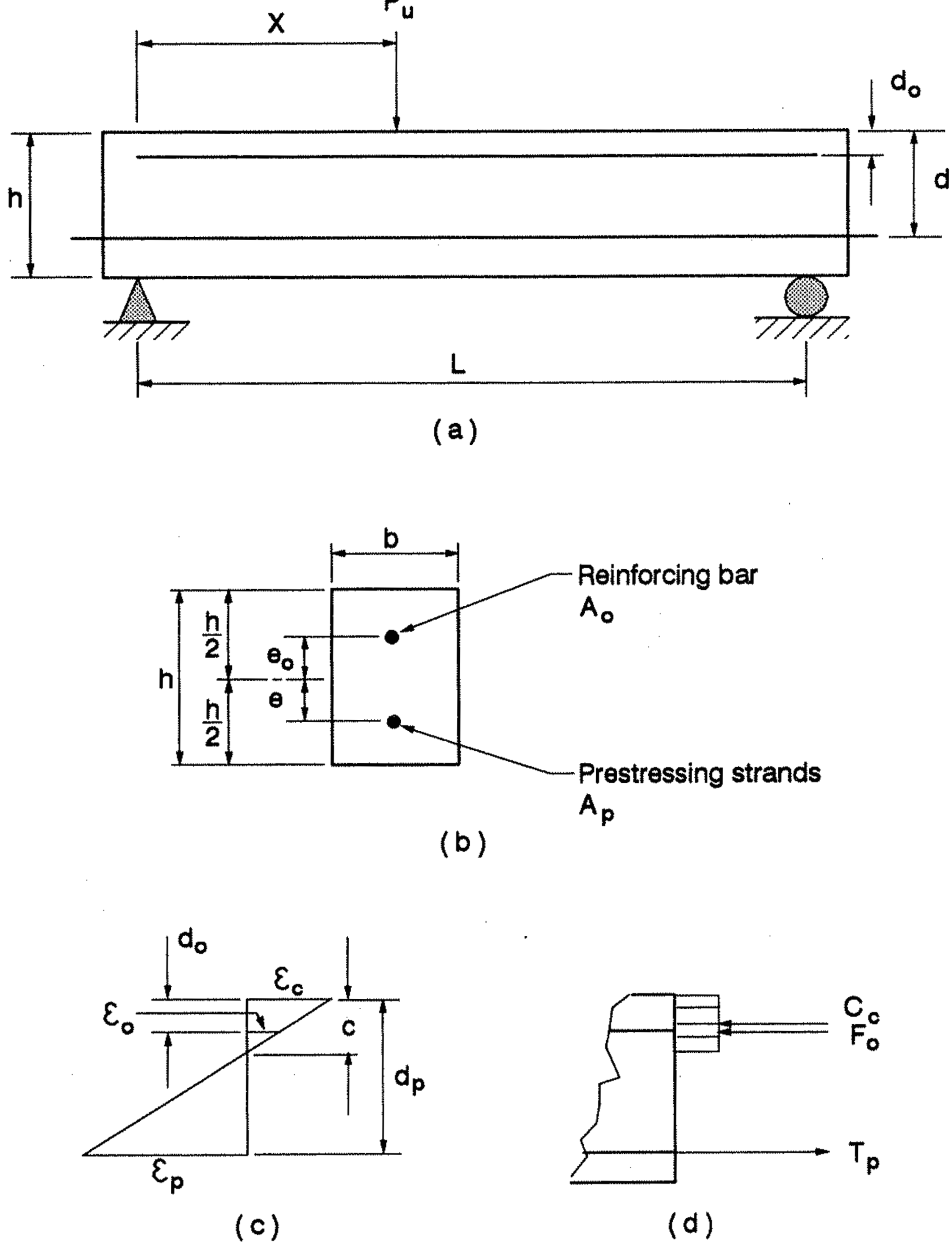


Figure 4.2. Nominal moment strength for a rectangular PC member: (a) elevation; (b) cross section at the load point; (c) bending strain distribution at the load point; (d) internal longitudinal forces at the load point

$$\epsilon_1 = \frac{f_{se}}{E_p} \tag{4.31}$$

where the stress f_{se} is the strand stress after prestress losses have occurred. As shown in Eq. (4.1), one of these losses is ES of the concrete at the location of the centroid of the prestressing strands. The longitudinal deformation of the member at this point in the cross section is caused by the axial prestress force and the induced bending moments produced by the eccentric strand position and the self-weight of the member. For the strand development length tests conducted in this research, only the ES losses were considered, since the other prestress losses were essentially nonexistent due to the short time interval that elapsed between prestressing and testing of the specimens.

The second load stage involves the decompression of the concrete at the centroid of the prestressing strands. A transverse load will induce a bending moment that causes a reduction of the concrete compressive strains in the precompressed region of the cross section. The end of the second loading stage occurs when the magnitude of the concrete strain at the strand centroid has been reduced to zero. If the prestressing strands do not slip relative to the surrounding concrete, the reduction in the concrete strain must be equal to the increase in the strand strain. Therefore, the additional strand strain ϵ_2 associated with the decompression of the concrete can be written as

$$\epsilon_2 = \left(\frac{1}{E_{cd}}\right)\left(\frac{T_e}{A_c} + \frac{T_e e^2}{I_g} - \frac{M_g e}{I_g}\right) \tag{4.32}$$

where the effective strand force T_e is given by

$$T_e = (f_{se})(A_p) \tag{4.33}$$

and E_{cd} = modulus of elasticity of the concrete when the transverse loads are applied to the member, A_c = cross-sectional area of the concrete, I_g = gross moment of inertia of the cross section with respect to the axis of bending and without considering the steel reinforcement, M_g = moment caused by the self-weight of the member, and A_p = total cross-sectional area of the prestressing strands. For the strand development length test specimens (D-type specimens) conducted in this research the strand strain ϵ_2 was almost identical to the elastic shortening strain loss when the specimens were prestressed. The strain difference was caused by the change in the modulus of elasticity of the concrete between prestressing and testing of the D-type specimens.

The third load stage involves transverse loadings that produce an additional strand strain ϵ_3 that is equal to the increase in the magnitude of the strand strain from the value at the end of the concrete decompression stage to the strain that occurs at the nominal moment strength of the member. By appling the linear strain distribution assumption to this load stage when the strength M_n is obtained, the additional strand strain ϵ_3 can be expressed as

$$\epsilon_3 = \epsilon_c \left(\frac{d_p - c}{c} \right) \tag{4.34}$$

where ϵ_c = maximum concrete strain (ϵ_c = 0.003 in./in.) that is associated with the strength M_n.

The total strain in the prestressing strands can now be obtained as

$$\epsilon_p = \epsilon_1 + \epsilon_2 + \epsilon_3 \tag{4.35}$$

Once the total strand strain ϵ_p is determined, the stress in the prestressing strands can be obtained from the stress-versus strain curve for low-relaxation strands. The PCI Design Handbook [41] provides two expressions for the strand stress that are based on elastic or inelastic behavior of the stress versus strain relationship. For elastic behavior, $\epsilon_p \leq 0.0086$ in./in. and the strand stress f^*_{su} is expressed as

$$f^*_{su} = E_p \, \epsilon_p \tag{4.36}$$

For inelastic behavior, $\epsilon_p > 0.0086$ in./in. and the strand stress f^*_{su} is given by

$$f^*_{su} = f'_s - \left(\frac{0.04}{\epsilon_p - 0.007} \right) \tag{4.37}$$

After the strand stress f^*_{su} has been established by either Eq. (4.36) or Eq. (4.37), the strand tension force T_p is evaluated as

$$T_p = f^*_{su} \, A_p \tag{4.38}$$

The strain ϵ_o in the nonprestressed reinforcement that was located near the compression face of the cross section can be established by using the linear strain distribution condition as

$$\epsilon_o = \epsilon_c \left(\frac{c-d_o}{c} \right) \tag{4.39}$$

The stress f_o in the nonprestressed steel is dependent on the strain level. If the steel does not yield ($|\epsilon_o| < \epsilon_y$), the stress f_o is expressed by

$$f_o = E_s\, \epsilon_o \tag{4.40}$$

and if the steel yields ($|\epsilon_o| \geq \epsilon_y$), the stress f_o is evaluated as

$$f_o = f_y \tag{4.41}$$

where ϵ_y = yield strength of the nonprestressed steel. Neglecting the effect of the concrete that is displaced by the steel bars when $\beta_1 c \geq d_o$, the force F_o in the nonprestressed reinforcement is obtained from

$$F_o = f_o A_o \tag{4.42}$$

where A_o = cross-sectional area of the nonprestressed reinforcement.

By utilizing the Whitney Rectangular Stress Block, the compression force in the concrete C_c is given by

$$C_c = 0.85\ f'_{cd}\ \beta_1 cb \tag{4.43}$$

The forces T_p, F_0, and C_c must satisfy the equilibrium equation

$$T_p = F_o + C_c \tag{4.44}$$

By a trial-and-error iteration approach, the correct neutral axis position that satisfies strain compatibility can be obtained. After the distance c has been established, the nominal moment strength of the PC member can be determined by

$$M_n = F_o\,(d_p - d_o) + C_c\left(d_p - \frac{\beta_1 c}{2}\right) \tag{4.45}$$

4.6. Nominal Shear Strength

The nominal shear strength V_n of a PC beam is equal to the sum of the nominal shear strength V_c provided by the concrete and the nominal shear strength V_s provided by the web reinforcement. When shear reinforcement is not provided, the shear strength V_n is the nominal concrete shear strength. The strength V_c is affected by the type of shear cracking that develops in a particular PC member. A flexure-shear crack develops from an initial flexural crack, and a web-shear crack develops in a beam that does not contain any previous flexural cracks [44]. Web-shear cracks usually occur only near the supports of beams that have thin webs; therefore, this type of shear failure is not common for a rectangular cross section. The concrete cracks that developed during the testing of the D-type specimens for this research revealed that only flexure-shear cracks occurred. Therefore, the concrete strength V_c will be evaluated considering flexure-shear crack resistance that is expressed by the ACI Building Code [3] Eq. (11-11) and AASHTO Specification [1] Eq. (9-27) and rewritten here as

$$V_n = \left(0.6\sqrt{f'_{cd}}\, bd_p + V_d + \frac{V_i M_c}{M_{max}}\right) \geq 1.7\sqrt{f'_{cd}}\, bd_p \tag{4.46}$$

with $d_p \geq 0.8h$ and where V_d = service level shear force due to dead loads, V_i = factored level shear force corresponding to the loading condition that produces the maximum factored level moment M_{max} and M_c = concrete cracking moment. The moment M_c can be approximated as

$$M_c = \frac{I_g}{Y_t}\left(6\sqrt{f'_{cd}} + f_{pe} - f_d\right) \tag{4.47}$$

where Y_t = distance from the centroid of the uncracked concrete cross section without reinforcement to the extreme tension fiber of the cross section when the external loads are applied; f_{pe} = concrete compressive stress, due to only the effective prestress force, at the location of the extreme tension fiber caused by the application of the external loads; and f_d = service level dead load stress at the location of the extreme tension fiber caused by the application of the external loads. When the ultimate concentrated transverse load P_u is applied at a distance X from the end of a rectangular-shaped, PC beam that has a uniform self-weight w_d, the internal shear forces V_d and V_i, moment M_{max}, and stresses f_{pe} and f_d are given by

$$V_d = \frac{1}{2} w_d (L - X) \tag{4.48}$$

$$V_i = \left[\frac{L - X}{L}\right] P_u \tag{4.49}$$

$$M_{max} = \left[\frac{X(L-X)}{L}\right] P_u \tag{4.50}$$

$$f_{pe} = T_e \left(\frac{1}{A_g} + \frac{eY_t}{I_g} \right) \tag{4.51}$$

$$f_d = \frac{\frac{1}{2}\, w_d\, X(L-X)\; Y_t}{I_g} \tag{4.52}$$

5. ANALYTICAL AND EXPERIMENTAL RESULTS

5.1. Material Properties

5.1.1. Concrete Tests

For each concrete casting, the concrete slump, air-entrainment, and compressive strengths were measured by applying the test procedures discussed in Section 3.4.3. The results of these tests are given in Table 5.1. The concrete batch size refers to the volume of concrete that was ordered from a ready-mix supplier. For some of the castings, water was added to obtain a more workable mix. As discussed in Section 3.4.3, the desired concrete slump was between 4 and 5 in. and the desired air entrainment was about 6 percent. Twelve of the seventeen castings had concrete slumps that did not exceed the preferred range by more than one inch. For most of the castings, the measured air content of the wet concrete mix was more than one percent away from the preferred amount. The variability in the concrete slump and air entrainment was attributed to the small quantities of concrete that were used in each of the castings. Table 5.1 also lists the concrete compressive strengths, f'_{ci}, f'_{cd}, and f'_c, that correspond to when the prestress forces were transferred to the specimens, when the strand development length tests were conducted, and when the concrete was 28-days old, respectively. The minimum concrete compressive strengths required by the Iowa DOT Specifications [21] for these three occurrences were 4000, 5000, and 5000 psi, respectively. Cast No. 6 was the only casting that did not obtain the required strength. As shown in the table, this casting had an exceptionally high amount of air entrainment. The concrete compressive strength for Cast No. 4 was quite high due to additional cement which was inadvertently added by the concrete supplier to the mix. The specimens from these two castings were not used in the determination of the strand transfer or development lengths.

Table 5.1. Measured concrete properties

Cast No.	Batch Size (yd^3)	Water added (gal.)	Slump (in.)	Air Entrainment (percent)	f'_{ci} (psi)	f'_{cd} (psi)	f'_c (psi)
1	1.5	8	4	5	4120	--	6430[a]
2	1.5	0	5	4	4780	--	6710
3	1.5	0	4	4	4710	--	7410
4	1.5	0	4	5	7240	--	9700
5	1.5	0	3	3	4640	--	6480
6	2.75	0	7	14	2910	2920	3620
7	2.75	3	2	6.5	3980	4890	6010[b]
8	3	0	6	7.5	4150	5120	5390[c]
9	3	3	6	4	4670	5710[e]	7200
10	2	0	5	3.5	4050	5350[e]	6480
11	2	0	5	3	4730	6150[e]	8020[d]
12	3	0	3	2.5	4420	6140	8080
13	2	1	5.5	3.5	4180	--	8430
14	3	12	7	5.5	4240	5230	6280
15	3	0	8	3	4010	5440	7280
16	3	18	4	2.5	4780	5870	7820
17	2	25	4	4	4390	5130	6780

[a]28-day strength linearly interpolated from 14-day and 33-day tests
[b]28-day strength linearly interpolated from 21-day and 34-day tests
[c]28-day strength linearly interpolated from 21-day and 35-day tests
[d]28-day strength linearly interpolated from 22-day and 35-day tests
[e]Average value for test duration

Figure 5.1 (a) and (b) show the specific data points of the concrete compressive strength versus time for the first 28 days and up to about 300 days, respectively. Except for two of the seventeen castings, the concrete strengths occurred with a banded region that varied from 5200 to 8500 psi at 28 days. The data points above and below this region are from Cast Nos. 4 and 6, respectively. These two concrete castings demonstrated that strength variability can occur when small quantities of concrete are ordered from a batch plant.

Concrete modulus of elasticity values for each casting were established by applying experimental results to analytical expressions. For the T-type specimens that were cast with embedment strain gages, the modulus of elasticity E_{ci} of the concrete, when the prestress force was applied to the concrete, was computed by

$$E_{ci} = \frac{\Sigma P_s - (A_p)(\epsilon_m)(E_p)}{(bh - A_p)\ \epsilon_m} \tag{5.1}$$

where ΣP_s = summation of the prestressing strand forces, A_p = total cross-sectional area of the prestressing strands in the section, ϵ_m = concrete strain beyond the strand transfer length that was measured immediately after strand release, E_p = modulus of elasticity of the prestressing strands, b = specimen width, and h = specimen thickness. This equation assumes that slippage between the strand(s) and the concrete does not occur at the location of the measured concrete strain. The strand development length tests for the D-type specimens provided another source for computing a modulus of elasticity for the concrete. Using the elastic portion of the load versus displacement relationship,

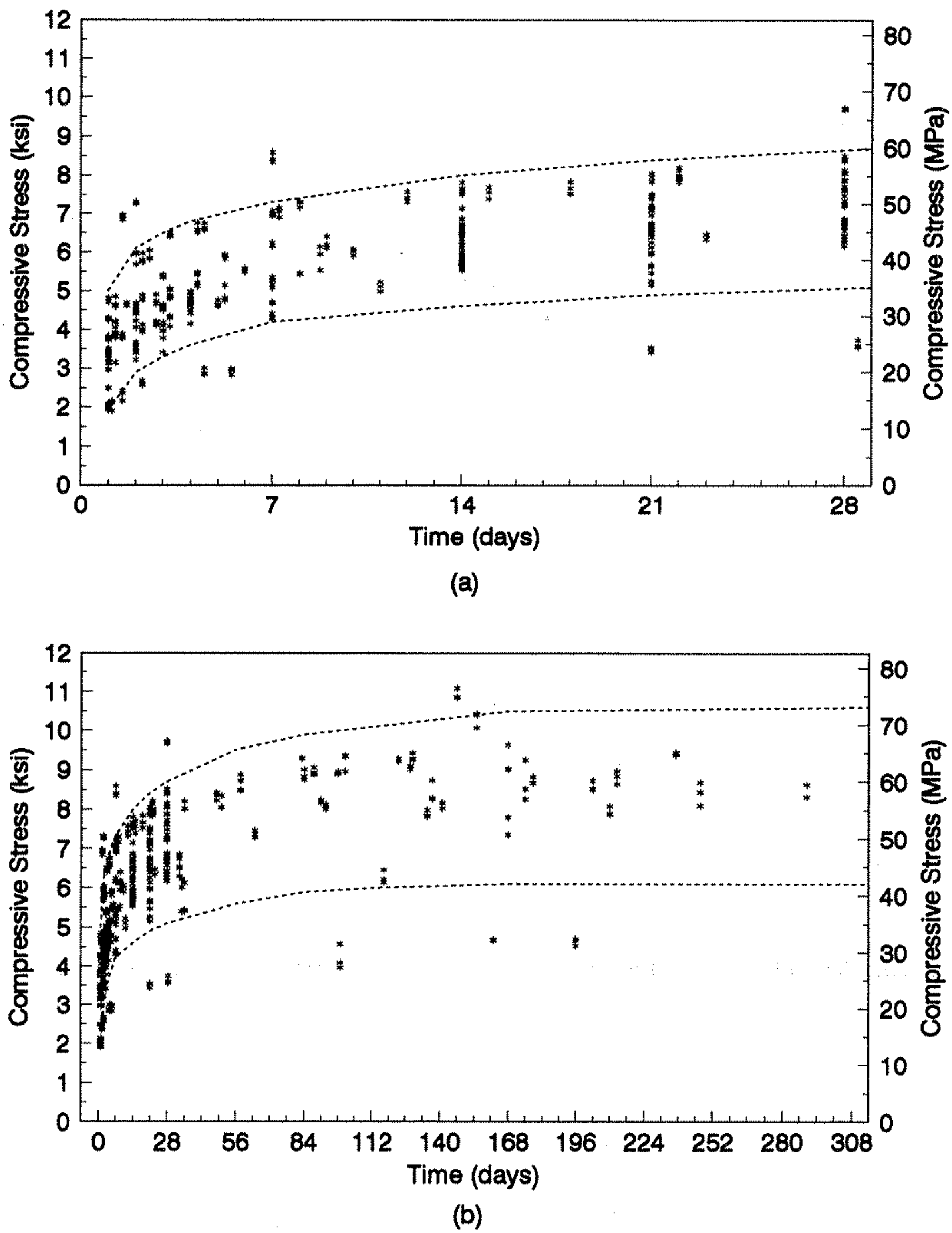


Figure 5.1. Concrete compressive strength versus time: (a) 0 to 28 days; (b) 0 to approximately 300 days

the modulus of elasticity, E_{cd}, of the concrete when transverse loads were applied to the specimens was calculated as

$$E_{cd} = \frac{PX^{2}(L-X)^{2}}{(3I_{g})(L)(\Delta_{p})} \tag{5.2}$$

where P = transverse elastic load, X = distance from the transverse load to the near end of the specimen, L = span length, I_g = gross moment of inertia of the cross section with respect to the axis of bending and without considering the steel reinforcement, and Δ_p = load-point deflection corresponding to the load P.

The empirical expressions that were used to calculate the modulus of elasticity of the normal-weight concrete were

$$E_{ci} = 57{,}000\sqrt{f'_{ci}} \tag{5.3}$$

$$E_{cd} = 57{,}000\sqrt{f'_{cd}} \tag{5.4}$$

and when the concrete was 28-days old, the modulus of elasticity, E_c, was computed by

$$E_{c} = 57{,}000\sqrt{f'_{c}} \tag{5.5}$$

Equation (5.5) is the ACI Building Code [4] expression for E_c. Equations (5.3) and (5.4), which have the same format as Eq. (5.5), were assumed to be applicable when the prestress force was transferred to the specimens and when the strand development length tests were conducted, respectively.

The modulus of rupture strengths, f_{ri} and f_r, of the concrete was experimentally established in accordance with ASTM C78 [9], by performing cross bending tests of 6-in. square beam prisms immediately after the T and D-type specimens were prestressed and when the concrete was about 28-days old, respectively. Also, the following emperical expressions were evaluated to establish the modulus of rupture strengths for the normal-weight concrete:

$$f_{ri} = 7.5\sqrt{f'_{ci}} \tag{5.6}$$

and

$$f_r = 7.5\sqrt{f'_c} \tag{5.7}$$

Equation (5.7) is the ACI Building Code [4] expression [Eq. (9.9)] for f_r. Equation (5.6), which has the same format as Eq. (5.7), was assumed to be applicable when the strands were released.

Table 5.2 lists the concrete modulus of elasticity values that were evaluated by Eqs. (5.1) through (5.5) and the modulus of rupture strengths that were experimentally measured from the beam prism tests and evaluated by Eqs. (5.6) and (5.7). Experimentally established values for both of these parameters can be significantly different from the values predicted from analytical expressions as discussed in Article 8.5.1 of the ACI Building Code [4] and in Section 1.8 from the textbook by Wang and Salmon [44]. The percent difference between the average experimental and the average predicted modulus of elasticity values was 16% immediately after the strands were released and when the strand development length tests were conducted. For these same two occurrences, the percent difference in the average modulus of rupture strengths was 8% and 25%, respectively.

Table 5.2. Concrete modulus of elasticity and modulus of rupture

Cast No.	Modulus of Elasticity ($x10^6$ psi)					Modulus of Rupture (psi)			
	At Release		At L_d Test		At 28 Days	At Release		At 28 Days	
	Test and Eq. (5.1)	Eq. (5.3)	Test and Eq. (5.2)	Eq. (5.4)	Eq. (5.5)	Test	Eq. (5.6)	Test	Eq. (5.7)
1	--	3.66	--	--	4.52	--	481	--	601
2	--	3.94	--	--	4.67	503	518	566	614
3	--	3.91	--	--	4.91	476	514	503[a]	646
4	--	4.85	--	--	5.61	483	638	496	739
5	--	3.88	--	--	4.59	475	511	452	604
6	--	3.07	2.72	3.08	3.43	323	405	387	451
7	--	3.60	3.64	3.99	4.36	421	473	437[b]	581
8	--	3.67	2.70	4.08	4.22	435	483	520[c]	551
9	--	3.90	3.09	4.31	4.84	404	512	467	636
10	3.87	3.63	4.31	4.17	4.59	497	477	524[d]	604
11	2.92	3.92	4.85	4.47	5.11	589	516	606[e]	672
12	4.06	3.79	3.60	4.47	5.12	487	499	424	674
13	--	3.69	--	--	5.23	544	485	511	689
14	2.89	3.71	2.94	4.12	4.52	443	488	530	594
15	2.76	3.61	3.77	4.20	4.86	421	475	529[a]	640
16	3.36	3.94	3.58	4.37	5.04	490	519	521	663
17	3.03	3.78	3.93	4.08	4.69	409	497	519	618
Avg.	3.27	3.80	3.56	4.12	4.72	462	499	500	622

[a]Test performed at 32-days
[b]Test performed at 31-days
[c]Test performed at 47-days
[d]Test performed at 33-days
[e]Test performed at 35-days

5.1.2. Strand Tests

Tension test results performed by the strand manufacturer indicated that the ultimate strength and modulus of elasticity of the 3/8-in. diameter, coated strand were 24.2 kips (corresponding to a stress of 285 ksi) and 28,900 ksi, respectively; and for the 3/8-in. diameter, uncoated strand, these parameters were equal to 23.8 kips (corresponding to a stress of 280 ksi) and 28,500 ksi, respectively. To confirm the reported modulus of elasticity values, tension test were performed during this research. Figures 5.2(a) and (b) show the stress versus strain relationships for the monitored coated and uncoated strands, respectively. A linear regression analysis of the generated data points was performed, and the computed modulus of elasticity was 29,600 ksi and 28,300 ksi for the coated and uncoated strands, respectively. As Fig. 5.2 shows, very consistent results were obtained for these tests. The percent differences from the manufacturer's modulus of elasticity results were 2.3% and 0.8% for coated and uncoated strands, respectively. The ultimate tensile strength of the strands were not experimentally verified, since special strand gripping devices that are required in order to obtain the fracture strength of a strand were not available.

5.2. Strand Force

Strand forces were monitored to obtain the proper prestressing force in a specimen. To determine the reason for the fluctuations in the strand forces that were observed during some of the initial castings, the researchers measured the temperatures of the air, steel frame, an exposed portion of a strand, and the concrete within a specimen during subsequent concrete castings. Figure 5.3 shows how temperature affects the prestress force in the six coated strands that were used in Cast No. 12. Before casting the concrete, the strand temperature was essentially equal to the laboratory

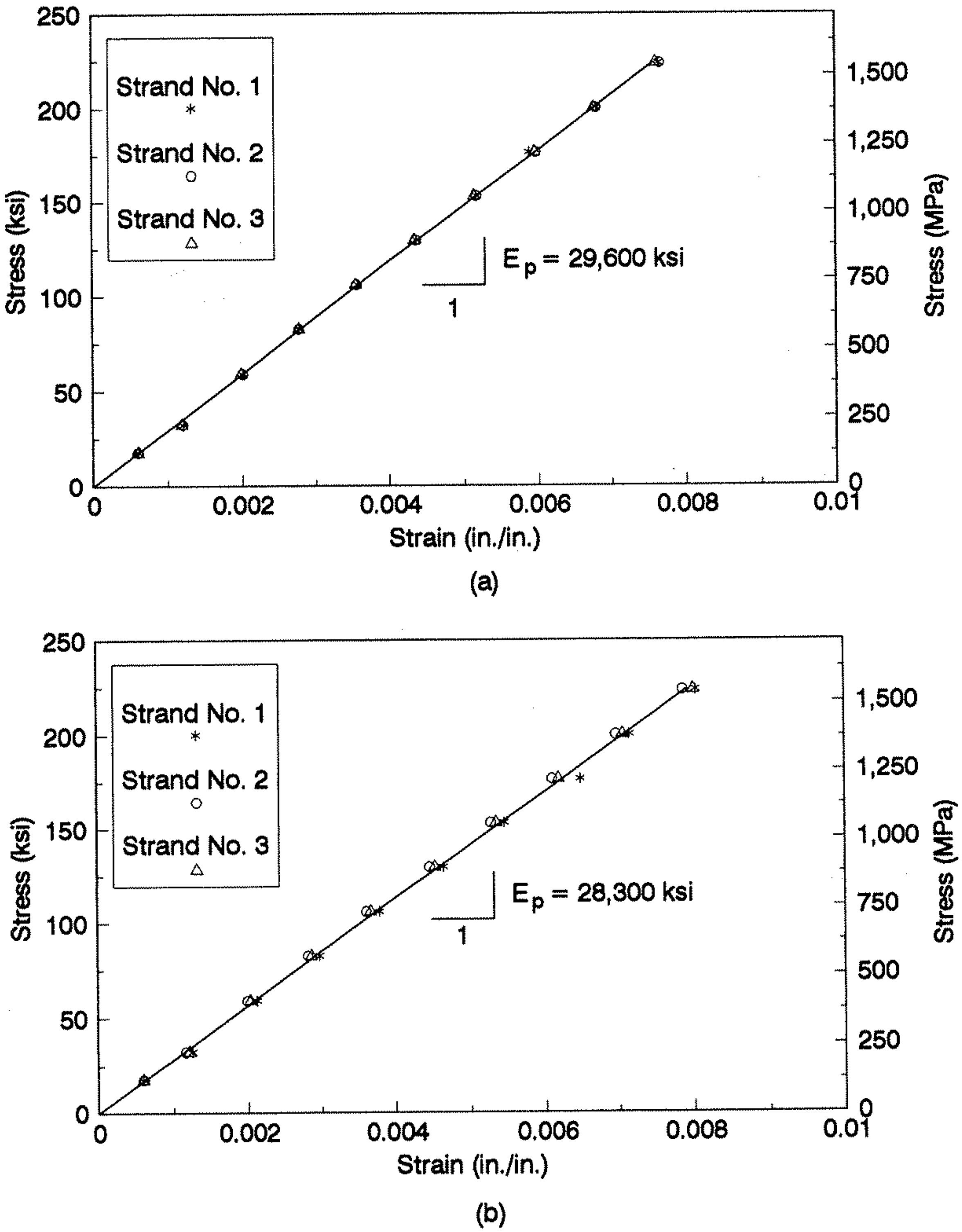
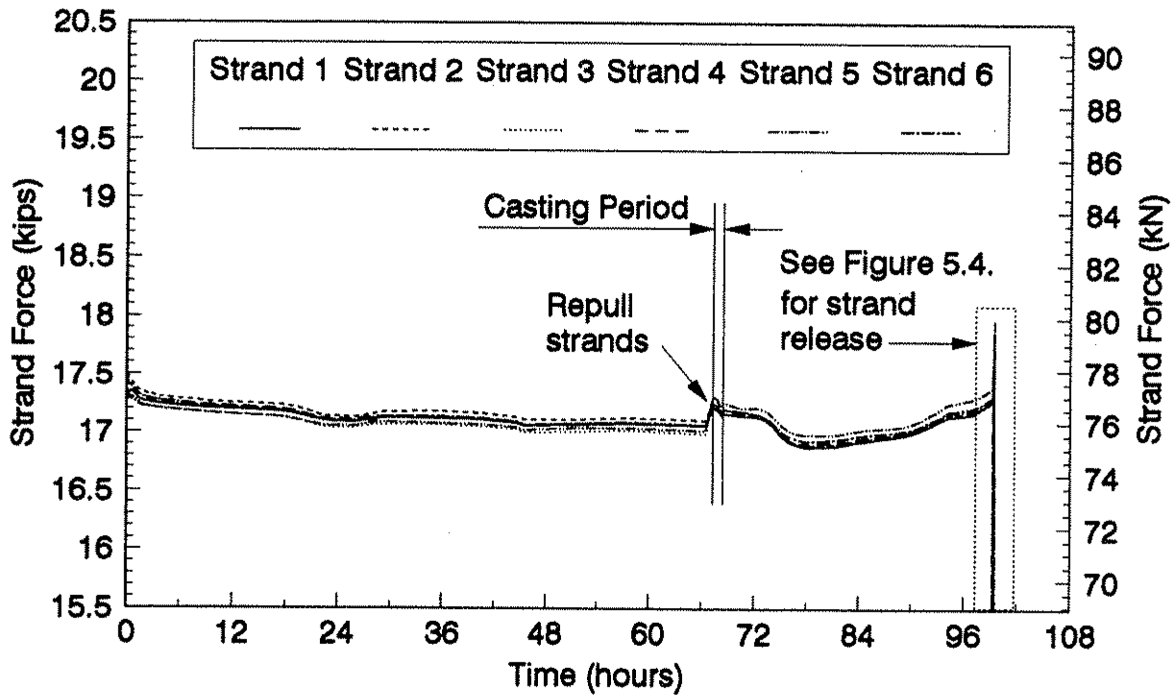
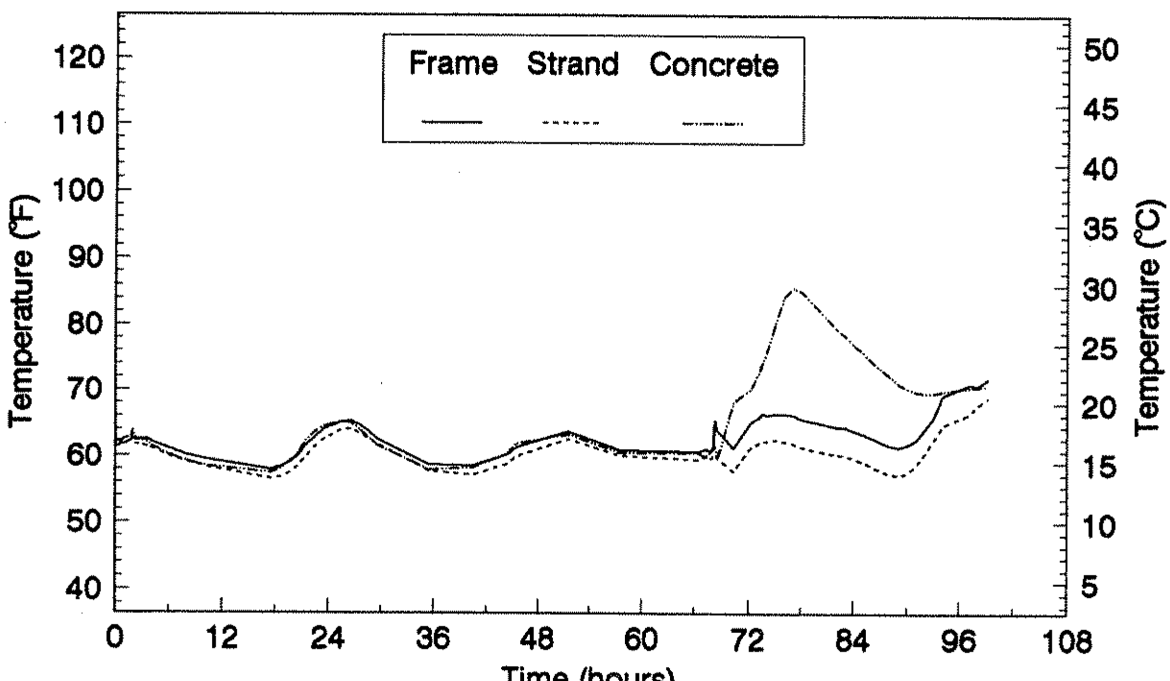


Figure 5.2. Initial stress versus strain curve for prestressing strands: (a) coated strand; (b) uncoated strand



(a)



(b)

Figure 5.3. Strand force and temperature variation for Cast No. 12: (a) strand force versus time; (b) temperature versus time

room temperature. During this time period, the prestressing force in the strands changed with time due to moderate room temperature fluctuations and relaxation of the strands since they were tensioned. After the concrete for the specimens was cast, the strand temperature was affected by the temperature of the surrounding concrete. The heat of hydration of the concrete caused the strand temperature to rise for about seven hours. This temperature increase produced a decrease in the prestressing force in the strands. Minimum strand forces occurred when the concrete temperature was a maximum. For Cast No. 12, the strand forces had decreased by about 400 lb each. As the concrete temperature decreased, the strand prestressing forces increased and approached their magnitudes present just prior to casting of the concrete. For Cast No. 12, Fig. 5.3(b) shows that the maximum concrete temperature recorded was about 86°F and the concrete temperature just prior to cutting the strands was about 72°F. For the nine concrete castings that had temperatures monitored, the maximum concrete temperature ever recorded was about 114°F and the maximum concrete temperature recorded just prior to cutting the strands was about 90°F. Both of these temperatures which occurred during Cast No. 16 were below the threshold temperature of 125°F that has been suggested by LeClaire and Shaikh [34] for members that are prestressed with coated strands. The temperature of the epoxy-coated strands when the strands are released is the critical temperature, since at that point the bond strength between the strands and the surrounding concrete is needed to transfer the prestress force to the concrete.

For each concrete casting, Table 5.3 lists the strand forces immediately before the concrete was cast and just prior to cutting the strands. During the process of prestressing the specimens, the forces in the strands changed. For Cast No. 12, Fig. 5.4 shows the variation in the strand forces during the progressive cutting of the strands. The notation along the abscissa scale indicates the

Table 5.3. Strand forces immediately before concrete casting and strand cutting

Cast No.	Strand Forces (kips)											
	Before Casting						Before Cutting					
	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6
1	17.18	17.12	17.12	17.05	17.14	17.08	17.22	17.32	17.20	17.17	17.37	17.10
2	17.29	17.24	17.29	17.32	17.34	17.35	17.40	17.32	17.36	17.31	17.33	17.35
3	17.26	17.25	17.25	17.26	17.23	17.20	17.30	17.28	17.28	17.29	17.25	17.25
4	17.26	17.21	17.25	17.17	17.23	17.18	17.23	17.19	17.21	17.13	17.21	17.00
5	17.21	17.25	17.23	17.30	17.26	17.27	17.28	17.28	17.28	17.32	17.28	17.32
6	17.38	17.36	17.25	17.40	17.26	17.27	17.19	17.06	16.92	17.11	17.01	17.00
7	17.26	17.30	17.19	17.27	17.24	17.28	17.31	17.34	17.20	17.30	17.28	17.33
8	17.21	17.33	17.23	17.32	17.25	17.31	17.15	17.25	17.17	17.25	17.18	17.24
9	17.31	17.34	17.32	17.33	17.31	17.34	17.32	17.25	17.26	17.25	17.27	17.28
10	17.31	17.30	17.32	17.32	----	----	17.29	17.25	17.23	17.24	----	----
11	17.41	17.45	17.43	17.39	----	----	17.24	17.23	17.28	17.27	----	----
12	17.22	17.24	17.25	17.24	17.30	17.24	17.28	17.27	17.29	17.29	17.39	17.31
13	18.94	19.03	19.04	19.05	18.87	19.01	18.86	18.92	18.93	18.95	18.81	18.98
14	17.35	17.21	17.39	17.27	17.37	17.22	17.36	17.13	17.39	17.36	17.37	17.26
15	17.18	17.22	17.23	17.21	17.23	17.18	17.08	17.13	17.12	17.12	17.16	17.11
16	17.22	17.18	17.20	17.22	17.16	17.23	17.23	17.20	17.21	17.24	16.79	17.25
17	17.19	17.26	17.25	17.26	----	----	17.20	17.20	17.17	17.27	----	----

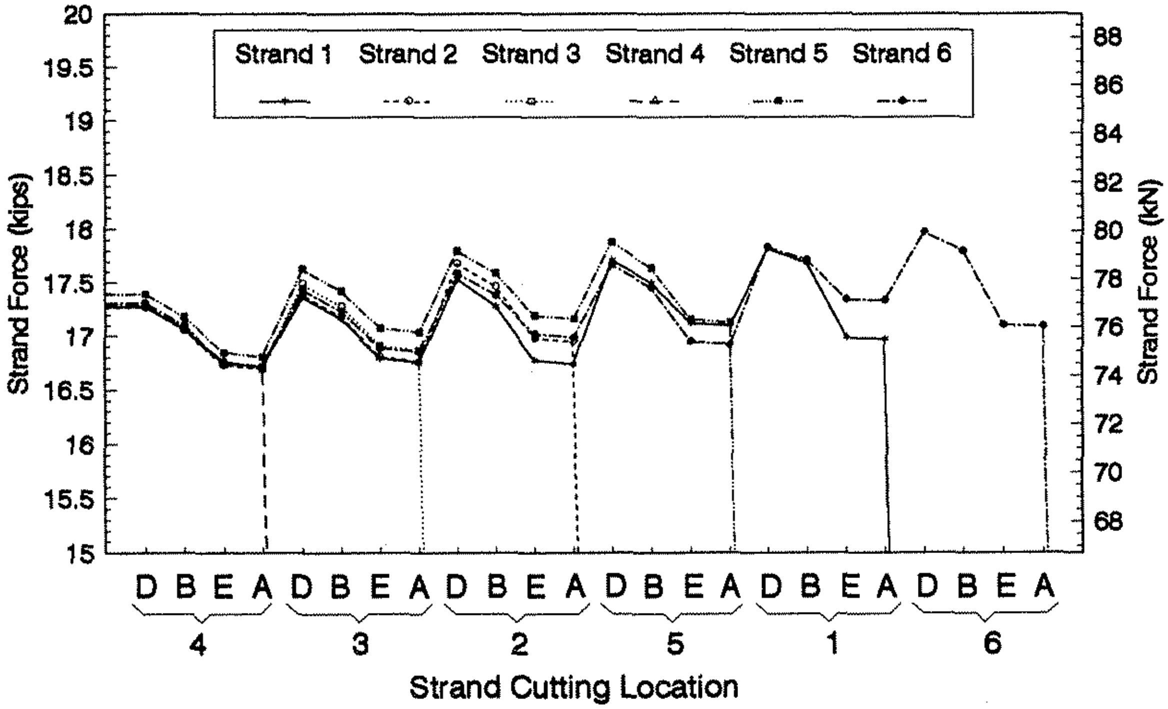


Figure 5.4. Strand force during cutting sequences for Cast No. 12

strand cutting sequence. The number represents the strand number and the letter corresponds to the prestressing frame single-header designation (see Fig. 3.1) at which a strand was cut. For example, the notation 4B signifies that a cut was performed on Strand No. 4 at Header B. The strand forces shown in Fig. 5.4 were measured at Header A. The strand force fluctuation is a characteristic of any prestress bed. For this research, the range of strand force variation was about one kip, representing about six percent of the initial strand prestress force. The range of force fluctuation would decrease with longer strands and more rigid anchorages in a prestress bed.

5.3. Minimum Panel Thickness

A total of 75 T-type specimens were tested for the purpose of establishing a recommended minimum thickness for PC deck panels containing coated strands and epoxy-coated WWF. Initial testing was conducted on 2.5 and 3.0-in. thick, T-type specimens that were 12 and 36-in. wide and contained coated prestressing strands. These specimens did not contain any WWF and the top surface was given a smooth finish. After prestressing these specimens, concrete cracks occurred only in the 2.5-in. thick specimens that had coated strands. The cracks were located at the ends of the specimens and were directly above and/or below some of the prestressing strands. Figure 5.5 shows the concrete cracks that developed in Specimen No. 3-2.5TC-10. The two concrete cracks that developed in this specimen occurred at the end of the specimen that was adjacent to Header D. The concrete crack at Strand No. 2 was 16-in. long in the top surface and 28-in. long in the bottom surface of the specimen. At Strand No. 1, a 14-in. long concrete crack developed only in the top surface of the specimen. The dimensions that locate a strand position were measured from the centroid of the strand to the face of the concrete or between the strand centroids. The accuracy of

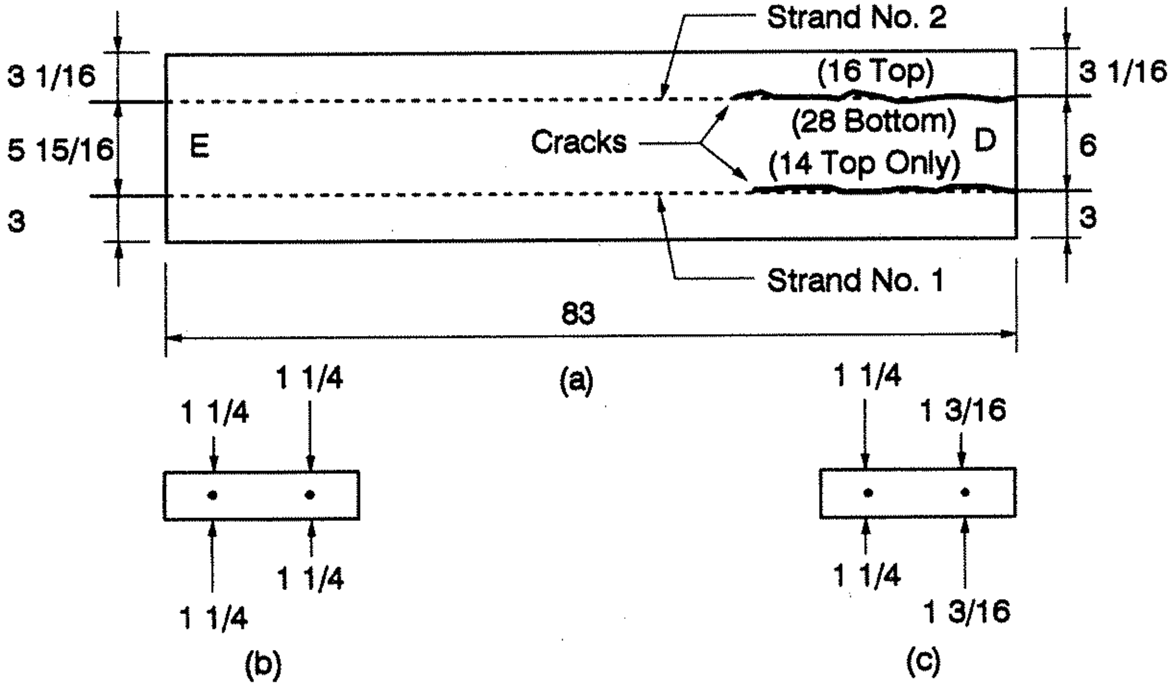


Figure 5.5. Specimen No. 3-2.5TC-10 (dimensions shown in inches): (a) plan view; (b) end view at E; (c) end view at D

these measurements were to the nearest 1/16 on an inch. The overall specimen length and the length of the concrete cracks in the top and bottom surfaces of the specimen have been rounded to the nearest whole inch. Measurements for all of the T-type specimens are provided in Appendix C. When the 3.0-in. thick specimens and the 2.5-in. thick specimens were prestressed with coated and uncoated strands, respectively, concrete cracks did not develop in any of these specimens. To observe if the presence of WWF in a specimen and a raked concrete surface on a specimen would affect whether the concrete would crack after prestressing a specimen, the researchers performed additional tests on both 2.5 and 3.0-in. thick T-type specimens that were 36-in. wide. As before, only the 2.5-in. thick specimens that were prestressed with coated strands developed concrete cracks. The presence of the WWF and a raked top concrete surface for the 3.0-in. thick specimens did not cause these specimens to develop cracks after the strands were cut.

The T-type specimens that experienced concrete cracking, the concrete crack location, and the concrete compressive strength f'_{ci} are given in Table 5.4. All of the specimens listed in Table 5.4 were 2.5-in. thick and prestressed with coated strands. Table 5.5 lists the T-type specimens that did not develop concrete cracks after the specimens were prestressed. This table lists the cast number, strand coating, concrete compressive strength f'_{ci}, nominal specimen size, number of specimens, and if both WWF was present in the specimens and a raked finish was provided to the top surface of the specimens. This table shows that only four 2.5-in. thick specimens that were prestressed with coated strands (two from Cast No. 2 and two from Cast No. 3) did not develop visible concrete cracks. On the basis of the results shown in Tables 5.4 and 5.5, a minimum thickness of 3 in. should be used for PC deck panels that are prestressed with coated strands.

Table 5.4. Cracked T-type specimens

Cast No.[a]	f'_{ci} (psi)	Specimen No.	Strand Nos. Contained in Specimen	Strand Nos. where Cracks were Detected		WWF[b] and Raked Top Surface
				End 1	End 2	
2	4780	2-2.5TC-2	3,4	--	4	No
	4780	2-2.5TC-3	5,6	--	5,6	No
	4780	2-2.5TC-10	1,2	1,2	1	No
	4780	2-2.5TC-11	3,4	4	--	No
3	4710	3-2.5TC-3	5,6	--	5,6	No
	4710	3-2.5TC-10	1,2	1,2	--	No
	4710	3-2.5TC-11	3,4	4	--	No
	4710	3-2.5TC-12	5,6	5	--	No
5	4640	5-2.5TC-1	1,2,3,4,5,6	--	1,6	No
	4640	5-2.5TC-2	1,2,3,4,5,6	1,6	6	No
	4640	5-2.5TC-3	1,2,3,4,5,6	6	1,6	No
	4640	5-2.5TC-4	1,2,3,4,5,6	1	1	No
8	4150	8-2.5TC-2	1,2,3,4,5,6	1,2,5,6	5,6	Yes

[a]Cast Nos. 2, 3, 5 and 8 contained only coated strands
[b]Epoxy-coated WWF

Table 5.5 Uncracked T-type specimens

Cast No.	Strand Coating[a]	f'_{ci} (psi)	Specimen Size (in.)		No. of Specimens	WWF[b] and Raked Top Surface
			h	b		
1	U	4120	2.5	12	3	No
	U	4120	3.0	12	3	No
	U	4120	3.5	12	3	No
	U	4120	4.0	12	3	No
2	C	4780	2.5	12	2	No
	C	4780	3.0	12	6	No
3	C	4710	2.5	12	2	No
	C	4710	3.0	12	6	No
4	C	7240[c]	3.0	12	12	No
6	U	2910[c]	3.0	36	1	No
7	U	3980	3.0	36	1	No
9	C	4670	3.0	36	1	Yes
10	U	4050	3.5	4	4	No
11	U	4730	3.0	6	4	No
12	C	4420	3.0	36	1	Yes
13[d]	C	4180	3.0	36	3	Yes
14	C	4240	3.0	36	1	Yes
15	U	4010	3.0	36	1	Yes
16	U	4780	3.0	36	1	Yes
17	C	4390	3.0	6	4	No

[a]C = Coated strand; U = Uncoated strand
[b]Epoxy-coated WWF used with coated strands and uncoated WWF used with uncoated strands
[c]Concrete compressive strength falls outside the banded range in Fig. 5.1
[d]Initial tension force for Cast No. 13 was 19.0 kips per strand

5.4. Strand Transfer Length

5.4.1. Experimental Results for Strand Transfer Length

Transfer lengths for coated and uncoated strands were determined from a graph of the concrete axial strains that were measured with PML-30 embedment strain gages. These internal gages were placed at incremental locations along the length and near the end of a type-T specimen that was adjacent to the location of the first cut on a particular strand. As the strands were cut, the concrete strains changed. The strand transfer lengths were established from the measured concrete strains immediately after the last strand cut was made and 18 hr. later. Only two of the ninety embedment gages installed for the monitored T-type specimens produced erroneous strain values. Except for Strand No. 1 in Specimen No. 15-3.0TU-2, the graphs of the strain distribution along the length of a specimen revealed two distinct regions of behavior. A region of increasing strain occurred at the end of a specimen, and a region of nearly constant strain existed along the interior of the length for a specimen. The strain results for Strand No. 1 in Specimen No. 15-3.0TU-2 revealed only a region of increasing strain, since the concrete was not completely consolidated around this edge strand. An essentially constant strain region indicated that the full effective prestress force had been transferred to the concrete.

A strand transfer length was calculated by using a slope-intercept method. This method involves averaging the strain readings in the region of constant strain to establish a best-fit horizontal line, applying a linear regression analysis to the strain values in the region of increasing strain to establish a best-fit sloping line through these data points, and computing the distance from the end of the specimen to the intersection point of the two straight lines. Figure 5.6 shows two sets of measured concrete axial strains at the embedded gage locations adjacent to Strand No. 4 in Specimen

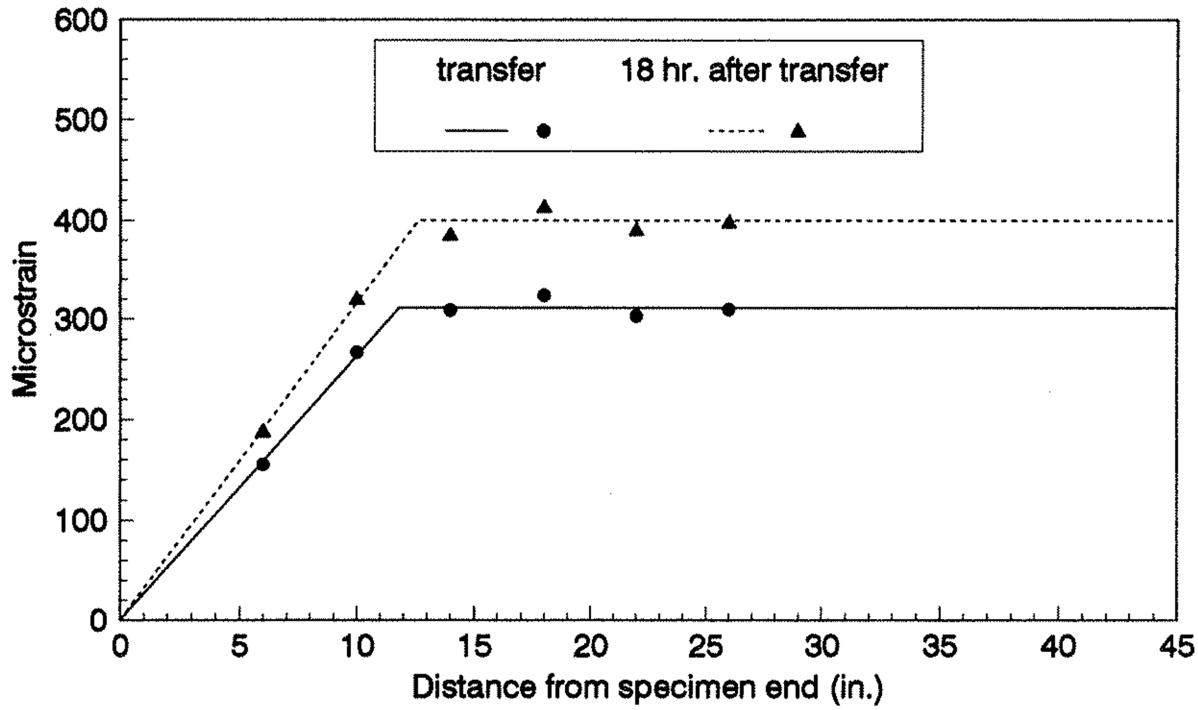


Figure 5.6. Concrete strains adjacent to Strand No. 4
in Specimen No. 14-3.0TC-2

No. 14-3.0TC-2 and the best-fit straight lines representing the induced concrete strains adjacent to this interior strand. The bi-linear behavior illustrated in Fig. 5.6 was also encountered with all of the other monitored strands in the other T-type specimens that had adequate consolidation of the concrete around the strands. The lower and upper concrete strain results shown in Fig. 5.6 are associated with the measurements that were taken just after the prestress force was transferred to the specimen and 18 hours after cutting the last strand, respectively. The two graphs for this specimen indicate that the concrete axial strain and the strand transfer length increased with time as a result of concrete creep. The increased concrete strain was noted for all of the 14 monitored strands in the T-type specimens. However, the strand transfer length increased slightly with time for only seven of these strands and either remained the same or actually decreased slightly with time for the other seven strands. Figures showing the results for the other strand transfer length tests are given in Appendix E.

The measured transfer length parameters for specimens containing coated and uncoated strands are listed in Tables 5.6 and 5.7, respectively. For a particular strand, the midlength concrete strain corresponds to the established constant axial strain. The initial strand prestress f_{si} was computed by dividing the particular strand force (see Table 5.3), which was measured before cutting any of the strands, by the cross-sectional area of the strand that was equal to 0.085 in.^{2} Assuming that a strand does not slip relative to the concrete in the region of constant axial strain, the prestress losses due to elastic shortening and creep of the concrete were computed by applying Hook's Law. Therefore, the measured concrete strain ϵ_m was equal to the change in the strain in the strand. The

Table 5.6. Measured transfer length parameters for specimens reinforced with coated strands

Specimen No.	Specimen Size (in.)		Midlength Concrete Strain (Microstrain)		f'_{ci} (psi)	f_{si} (ksi)	Prestress Losses (ksi)		f_{se} (ksi)		L_t (in.)	
	b	h	At Release	At 18 Hours			At Release	At 18 Hours	At Release	At 18 Hours	At Release	At 18 Hours
17-3.0TC-6	6	3.0	298	395	4390	202.3	8.8	11.7	193.5	190.6	11.8	10.9
17-3.0TC-7	6	3.0	292	402	4390	202.0	8.6	11.9	193.4	190.1	10.0	11.0
17-3.0TC-8	6	3.0	301	412	4390	203.1	8.9	12.2	194.2	190.9	11.8	12.3
12-3.0TC-2	36[a]	3.0	207	249	4420	203.2	6.1	7.4	197.1	195.8	21.1	21.4
	36[b]	3.0	245	304	4420	203.4	7.3	9.0	196.1	194.4	18.6	18.4
14-3.0TC-2	36[a]	3.0	327	405	4240	204.3	9.7	12.0	194.6	192.3	11.1	11.6
	36[b]	3.0	317	400	4240	204.2	9.4	11.8	194.8	192.4	11.8	12.7
Average	-	-	-	-	4356	203.2	8.4	10.9	194.8	192.4	13.7	14.0

[a]Strand No. 1 (edge strand)
[b]Strand No. 4 (interior strand)

Table 5.7. Measured transfer length parameters for specimens reinforced with uncoated strands

Specimen No.	Specimen Size (in.)		Midlength Concrete Strain (Microstrain)		f'_c (psi)	f_{si} (ksi)	Prestress Losses (ksi)		f_{se} (ksi)		L_t (in.)	
	b	h	At Release	At 18 Hours			At Release	At 18 Hours	At Release	At 18 Hours	At Release	At 18 Hours
10-3.5TU-6	4	3.5	432	499	4050	202.9	12.2	14.1	190.7	188.8	33.1	27.4
10-3.5TU-7	4	3.5	382	490	4050	202.7	10.8	13.9	191.9	188.8	37.9	35.4
11-3.0TU-6	6	3.0	294	391	4730	202.7	8.3	11.1	194.4	191.6	23.6	24.3
11-3.0TU-7	6	3.0	300	405	4730	203.3	8.5	11.5	194.8	191.8	25.6	24.8
15-3.0TU-2	36[a]	3.0	373	447	4010	201.0	10.6	12.7	190.4	188.3	39.3[c]	36.7[c]
	36[b]	3.0	293	365	4010	201.4	8.3	10.3	193.1	191.1	26.7	30.3
16-3.0TU-2	36[a]	3.0	273	346	4780	202.6	7.7	9.8	194.9	192.8	19.7	19.3
	36[b]	3.0	273	344	4780	202.9	7.7	9.7	195.2	193.2	16.4	15.9
Average	-	-	-	-	4393	202.4	9.3	11.6	193.2	190.8	27.8	26.8

[a]Strand No. 1 (edge strand)
[b]Strand No. 4 (interior strand)
[c]Long length due to incomplete consolidation of the concrete along the exterior strand

prestress losses PL were computed as

$$PL = \epsilon_m E_p \tag{5.8}$$

where the modulus of elasticity for the strand E_p was experimentally established as discussed in Section 5.1.2. The effective strand prestress f_{se} immediately after strand release was computed by

$$f_{se} = f_{si} - PL \tag{5.9}$$

The coated and uncoated strand transfer lengths listed in Tables 5.6 and 5.7, respectively, were computed by applying the slope-intercept method. For the 6 and 36-in. wide specimens, the transfer lengths for the coated strand ranged from 10.0 to 21.1 in. immediately after strand release and from 10.9 to 21.4 in. eighteen hours later. The uncoated strand transfer lengths ranged from 16.4 to 26.7 in. immediately after strand release and from 15.9 to 30.3 in. 18 hours later. The large range in the strand transfer lengths was assumed to be caused by the amount of concrete confinement, degree of concrete consolidation that was present around a monitored strand, the concrete strength f'_{ci}, and the speed of cutting the strand during strand release for the particular T-type specimen. The measured, coated-strand transfer lengths for Specimen No. 12-3.0TC-2 from Cast No. 12 are long compared to the lengths obtained for the other specimens from Cast Nos. 14 and 17. Table 5.1 has shown that both the concrete slump and amount of air entrainment were smaller for Cast No. 12 than for Cast Nos. 14 and 17. Also, water had not been added to the concrete mix for Cast No. 12, while water had been added to the other two concrete castings. Therefore, the concrete in Cast No. 12 might not have properly consolidated around the strands within the transfer region in Specimen No. 12-3.0TC-2, causing the longer strand transfer lengths to occur for this specimen. Even though the

18.6 and 21.1-in. transfer lengths that occurred immediately after the strands were cut appear not to be totally representative, these lengths will still be used to provide an upper bound result for the average, coated-strand transfer lengths for the 36-in. wide specimens. The measured, coated-strand transfer lengths for the 6-in. wide specimens were almost the same as those lengths measured for Specimen No. 14-3.0TC-2 which was 36-in. wide. Therefore, the 6-in. strand spacing in the 36-in. wide specimen did not appear to affect the coated-strand transfer lengths.

As shown in Table 5.7, the measured, uncoated-strand transfer lengths for Specimen No. 16-3.0TU-2 from Cast No. 16 are short compared to the lengths obtained for the 6-in. wide specimens and the interior strand for the other 36-in. wide specimen. A comparison of the concrete properties given in Table 5.1 for Cast Nos. 10, 11, 15, and 16 does not provide an explanation as to why Specimen No. 16-3.0TU-2 had the shorter strand transfer lengths. The uncoated-strand transfer lengths for the edge strand in Specimen No. 15-3.0TU-2 were large and not totally representative due to the incomplete consolidation of the concrete that was observed in the form of some honey-combing along the edge of this specimen. However, to produce an upper bound on the uncoated-strand transfer length, this result will be applied. The 6-in. strand spacing in one of the 36-in. wide specimens did not appear to affect the uncoated-strand transfer length.

The average of the measured, coated and uncoated-strand transfer lengths for the monitored T-type specimens and the ratio of these average lengths are given in Table 5.8. Depending on the specimen size, the transfer lengths for the coated strands were significantly shorter than the transfer lengths for the uncoated strands. The influence of the concrete edge distance on the uncoated-strand transfer length was established by noting that the average transfer length of the two uncoated interior strands was shorter than the average transfer length for the two uncoated edge strands. A shorter transfer length for an interior uncoated strand with adequate spacing compared to that for an edge strand with a limited concrete edge distance is reasonable, since the interior strand has a sufficient amount of concrete on each side of the strand to resist a concrete splitting failure. Even though the average transfer length for the two coated interior strands was less than the average strand transfer length for the two coated edge strands, the actual influence of the concrete edge distance for coated strands could not be established because the coated-strand transfer lengths for the individual specimens were not always longer for an edge strand, as shown in Table 5.6. This behavioral difference was caused by the different bond mechanisms for a coated strand and an uncoated strand, as discussed in Section 4.1.

5.4.2. Comparisons with Other Researchers

The parameters that affect the transfer length of a prestressing strand are the concrete cover, strand spacing, strand surface coating, concrete compressive strength at the time of strand release, effective strand prestress, method of strand release (sudden or gradual), and the amount of time that has elasped since force transfer. Tables 5.9 and 5.10 list the experimentally derived, transfer length measurements for coated and uncoated strands, respectively, that were obtained by this and other research. A comparison of the strand transfer lengths for specimens having similar parameters reveals that good agreement was obtained by the researchers listed in these tables.

To determine whether analytical models predicted the transfer lengths for coated and uncoated strands used in this research, the researchers compared the measured and calculated strand transfer lengths. The empirical expressions that were applied for the strand transfer length have been taken from the following references to formulate the equations noted here: Cousins et al. [14] for Eq.

Table 5.8. Average measured transfer lengths for coated and uncoated strands

Specimen Size (in.) b	Specimen Size (in.) h	Strand Coating[a]	No. of Specimens	Average L_t (in.) At Release	Average L_t (in.) At 18 Hours	Coated L_t / Uncoated L_t At Release	Coated L_t / Uncoated L_t At 18 Hours
4	3.5	U	2	35.5	31.4	-	-
6	3.0	C	3	11.2	11.4	0.46	0.46
	3.0	U	2	24.6	24.6		
36	3.0	C[b]	2	16.1	16.5	0.55	0.59
36	3.0	U[b]	2	29.5	28.0		
36	3.0	C[c]	2	15.2	15.6	0.70	0.68
36	3.0	U[c]	2	21.6	23.1		

[a]C = coated strand; U = uncoated strand
[b]Strand No. 1 (edge strand)
[c]Strand No. 4 (interior strand)

Table 5.9. Comparisons of measured transfer lengths for 3/8-in. diameter, seven-wire, low-relaxation, coated strands

Researcher	Strand Type	Specimen Size (in.) b	Specimen Size (in.) h	Avg. f'_{ci} (psi)	Avg. f_{se} (ksi)	Average L_t (in.) At Release	Average L_t (in.) After Release
Abendroth, Stuart, Yuan	270-ksi Low-Relaxation	6 36	3.0 3.0	4390 4330	193.9 195.8	11.2 15.7	11.4[a] 16.1[a]
Cousins, Johnston, Zia [12,14]	270-ksi Low-Relaxation	3.5	3.5	4190	187.2	-	13.0[b] 15.0[c]
FHWA as reported by Lane [31]	270-ksi Low-Relaxation	4.0 to 9.0	4.0 to 9.0	4330	Max. 202.5	-	19.2[c]

[a]Measured 18 hr. after transfer
[b]Measured 1 day after transfer
[c]Measured 1 year after transfer

Table 5.10. Comparisons of measured transfer lengths for 3/8-in. diameter, seven-wire, uncoated strands

Researcher	Strand Type	Specimen Size (in.)		Avg. f'_{ci} (psi)	Avg. f_{se} (ksi)	Average L_t (in.)	
		b	h			At Release	After Release
Abendroth, Stuart, Yuan	270-ksi Low-Relaxation[f]	4	3.5	4050	191.2	35.5	31.4[a]
		6	3.0	4730	194.5	24.6	24.6[a]
		36	3.0	4400	193.3	25.6	25.6[a]
Abendroth, Pratanata, Singh [2]	270-ksi Low Relaxation[f]	96	2.5	4810	195	-	24.0[b]
Cousins, Johnston, Zia [12,14]	270-ksi Low-Relaxation[f]	3.5	3.5	4190	184.4	-	34.8[b] 36.8[d]
Kaar, LaFraugh, Mass [25]	250-ksi Stress-Relieved[g]	4.5	6.3	1690	176.8	22.2	-
		4.5	6.3	3400	168.6	27.0	-
		4.5	6.3	5020	158.8	23.5	-
FHWA as reported by Lane [32]	270-ksi Low-Relaxation[f]	4.0 to 9.0	4.0 to 9.0	4330	Max. 202.5	-	26.9[d]
Mitchell, Cook, Khan, Tham [37]	270-ksi Low-Relaxation[f]	3.9	7.9	3000	177	19.9	19.0[c]
		3.9	7.9	3975	180	19.0	23.0[c]
		3.9	7.9	6950	173	11.9	-
		3.9	7.9	7225	178	14.0	14.0[c]
		3.9	7.9	7310	179	16.4	16.4[c]
Ban, Muguruma, Morita [11]	240-ksi Stress-Relieved[g]	3.9	3.9	4096	148	15.7	-
		3.9	3.9	6187	148	13.8	-
		3.1	3.1	4068	148	15.7	-
		2.4	2.4	4082	148	15.7	-
Over, Au [39]	250-ksi Stress-Relieved[g]	3.0	3.0	4180	150[h]	30.0[e]	

[a]Measured 18 hr. after transfer
[b]Measured 1 day after transfer and adjusted to slope intercept definition
[c]Measured 21 days after transfer
[d]Measured 1 year after transfer
[e]Age of specimen at time of strand release was not specified
[f]Strand area equals 0.085 in.^{2}
[g]Strand area equals 0.080 in.^{2}
[h]Interpolated stress

(4.15), the PCI Ad Hoc Committee on Epoxy-Coated Strand (PCI Guidelines) [40] for Eq. (4.9), the AASHTO Specifications [1] and ACI Building Code [3] for Eq. (4.8), Zia and Mostafa [45] for Eq. (4.10), and Mitchell et al. [37] for Eq. (4.14). Table 5.11 lists the measured, coated-strand transfer lengths that were established immediately after prestressing each of the monitored T-type specimens that contained this type of strand; the specimen size; the stresses f'_{ci}, f_{si}, and f_{se}; and the calculated strand transfer lengths based on these experimentally obtained stresses and the expressions given by references [14, 40, 1, and 3] above. The table shows that the AASHTO and ACI predicted L_t-lengths were always greater than the measured L_t-lengths for the monitored coated strands, while except for one specimen, the coated strand transfer lengths established by Cousins et al. and the PCI Guidelines were conservative. For Strand No. 1 (an edge strand) in Specimen No. 12-3.0TC-2, the measured L_t-length was about 43% and 12% greater than the length predicted by the expression given by Cousins et al. and the PCI Guidelines, respectively. The average of the measured, coated-strand transfer lengths was equal to 13.7 in. This length was overestimated by about 7%, 37%, and 78% by applying the equations given by Cousins et al., the PCI Guidelines, and implied in the AASHTO Specifications and given in the ACI Building Code, respectively.

The nominal, coated-strand transfer lengths were computed by substituting the nominal stresses into the expressions presented by Cousins et al. [14] and implied in the AASHTO Specification [1] and given in the ACI Building Code [3]. Thus, for f'_{ci} set equal to the minimum strength of 4000 psi and f_{se} established as 195.2 ksi from Eq. (4.13) with f'_s = 270 ksi, E_p = 28,500 ksi, and E_{ci} = 3605 ksi from Eq. (5.3), the nominal L_t-lengths for coated strands are 15.2 in. and 24.4 in., respectively. As shown in Table 5.11, these nominal lengths were close to the predicted strand transfer lengths.

Table 5.11. Comparisons of measured and calculated transfer lengths for 3/8-in. diameter, seven-wire, 270-ksi, low-relaxation, grit-impregnated, epoxy-coated strands

Specimen No.	Specimen Size (in.)		f'_{ci} (psi)	f_{si} (ksi)	f_{se} (ksi)	Strand Transfer Length at Release (in.)			
	b	h				Abendroth, Stuart, Yuan (Measured)	Cousins et al [14] (Eq. 4.15)[d]	PCI Guidelines [40] (Eq. 4.9)	AASHTO [1] & ACI [3] (Eq. 4.8)
12-3.0TC-2	36[a]	3.0	4420	203.2	197.1	21.1	14.8	18.8	24.6
	36[b]	3.0	4420	203.4	196.1	18.6	14.7	18.8	24.5
14-3.0TC-2	36[a]	3.0	4240	204.3	194.6	11.1	14.9	18.8	24.3
	36[b]	3.0	4240	204.2	194.8	11.8	14.9	18.8	24.4
17-3.0TC-6	6	3.0	4390	202.3	193.5	11.8	14.6	18.8	24.2
17-3.0TC-7	6	3.0	4390	202.0	193.4	10.0	14.6	18.8	24.2
17-3.0TC-8	6	3.0	4390	203.1	194.2	11.8	14.6	18.8	24.3
Average	-	-	4356	203.2	194.8	13.7	14.7[c]	18.8	24.4[c]
Nominal	6 or 36	3.0	4000	202.5	195.2	-	15.2	18.8	24.4

[a]Strand No. 1 (edge strand)
[b]Strand No. 4 (interior strand)
[c]Based on applying the average stresses
[d]U'_t = 16.5 medium to high grit density

Table 5.12 lists the measured, uncoated-strand transfer lengths that were established immediately after prestressing each of the monitored T-type specimens that contained this type of strand; the specimen size; the stresses f'_{ci}, f_{si}, and f_{se}; and the calculated strand transfer lengths based on the expressions given by Cousins et al. [14], implied in the AASHTO Specifications [1] and given in the ACI Building Code [3], presented by Zia and Mostafa [45], and given by Mitchell et al. [37]. The table shows that the predicted L_t-lengths that were obtained from the expression by Cousins et al. were conservative and that the predicted L_t-lengths established by applying the equations presented by Zia and Mostafa and Mitchell et al. underestimated the measured strand transfer lengths for most of the T-type specimens. The predicted strand transfer lengths obtained from the implied expression in the AASHTO Specification and the equation in the ACI Building Code underestimated the measured strand transfer lengths for the two 4-in. wide specimens, was quite accurate for the two 6-in. wide specimens, and was conservative for the two monitored strands in one of the 36-in. wide specimens and underestimated the measured strand transfer lengths for the two monitored strands in the other 36-in. wide specimen. The average of the measured, uncoated-strand transfer lengths was equal to 27.8 in. This length was overestimated by about 15% when the expression by Cousins et al. was applied, underestimated by about 15% when the expression implied AASHTO Specification and stated ACI Building Code was applied, underestimated by about 31% when the expression by Zia and Mostafa was applied, and underestimated by about 40% when the expression by Mitchell et al. was applied.

The nominal, uncoated-strand transfer lengths were computed by using nominal stresses in the expressions presented by Cousins et al. [14], Zia and Mostafa [45], and Mitchell et al. [37] and implied in the AASHTO Specification [1] and given in the ACI Building Code [3]. Using the same

Table 5.12. Comparisons of measured and calculated transfer lengths for 3/8-in. diameter, seven-wire, 270-ksi, low-relaxation, uncoated strands

Specimen No.	Specimen Size (in.)		f'_{ci} (psi)	f_{si} (ksi)	f_{se} (ksi)	Strand Transfer Length at Release (in.)				
	b	h				Abendroth, Stuart, Yuan (Measured)	Cousins et al [14] (Eq. 4.15)	AASHTO [1] & ACI [3] (Eq. 4.8)	Zia & Mostafa [45] (Eq. 4.10)	Mitchell et al [37] (Eq. 4.14)
10-3.5TU-6	4	3.5	4050	202.9	190.7	33.1	33.0	23.8	23.6	20.3
10-3.5TU-7	4	3.5	4050	202.7	191.9	37.9	33.2	24.0	23.6	20.4
11-3.0TU-6	6	3.0	4730	202.7	194.4	23.6	31.6	24.3	19.5	19.2
11-3.0TU-7	6	3.0	4730	203.3	194.8	25.6	31.3	24.4	19.6	19.2
15-3.0TU-2	36[a]	3.0	4010	201.0	190.4	39.3[c]	34.9	23.8	23.6	20.4
	36[b]	3.0	4010	201.4	193.1	26.7	35.0	24.1	23.7	20.7
16-3.0TU-2	36[a]	3.0	4780	202.6	194.9	19.7	32.3	24.4	19.2	19.1
	36[b]	3.0	4780	202.9	195.2	16.4	32.4	24.4	19.3	19.1
Average	-	-	4393	202.4	193.2	27.8	32.1[d]	24.1[d]	21.3[d]	19.8[d]
Nominal	4	3.5	4000	202.5	193.2	-	33.6	24.1	23.9	20.7
	6 or 36	3.0	4000	202.5	195.2	-	33.9	24.4	23.9	20.9

[a]Strand No. 1 (edge strand)
[b]Strand No. 4 (interior strand)
[c]Long length due to incomplete consolidation of the concrete around the edge strand
[d]Based on applying the average stresses

approach that was applied for coated strands, the researchers evaluated the nominal stresses f'_{ci} and f_{si} as 4000 psi and 202.5 ksi, respectively. The nominal stress f_{se} was established as 193.2 ksi for the 4-in. wide specimens and 195.2 ksi for the 6 and 36-in. wide specimens. The nominal, uncoated-strand transfer lengths were computed as 33.6 and 33.9, 24.1 and 24.4, 23.9 and 23.9, and 20.7 and 20.9 in. based on the expression by Cousins et al., AASHTO and ACI, Zia and Mostafa, and Mitchell et al., respectively, for the 4-in. wide specimens and 6 and 36-in. wide specimens, respectively. As shown in Table 5.12, these nominal lengths were close to the predicted strand transfer lengths, except for the lengths obtained by applying the expression by Zia and Mostafa to the higher strength concrete specimens. However, the nominal strand transfer length for those specimens was conservative.

5.5. Strand Development Length

5.5.1. Modes of Failure

As discussed in Section 3.4.6., the failure mechanism for a D-type specimen needed to be experimentally established so that the position for the transverse load could be adjusted until convergence to the end of the strand development length occurs. The three primary modes of failure for a laterally braced PC beam that is reinforced with prestressing strands are flexural, bond, and shear. These failure modes will be referred to as the primary failure mode types F, B, and S, respectively. A bond failure refers to slippage of a prestressing strand (strand-slip) at an end of the specimen. If the failure of a D-type specimen involved the apparent simultaneous formation of a bond failure in combination with either a flexural or shear failure, the order in which they occurred in a specimen was noted whenever possible. When a flexural mode of failure appeared to immediately induce a loss of bond strength, the failure mode is classified as a type F/B failure. Conversely, if the order of mechanism formation was reversed, the type of failure was termed a B/F mode of failure. Similarly, for combinations of shear and bond failures, the failure modes will be termed S/B and B/S. When a specimen failure involved both flexure and shear failures, the order of formation could not be distinguished; therefore, this type of failure will be referred to as an FS failure.

If the nominal shear strength for a specimen was greater than the maximum shear force induced in the specimen, a flexural failure occurred when the distance from the near end of a specimen to the transverse load position was larger than the strand development length. Figure 5.7 shows the load versus load-point deflection relationship at 54 in. from End E for Specimen 10-6.0DU-9. When the induced bending moment at a particular cross section exceeded the concrete cracking moment, flexural tension cracks began to become visible on the side of the specimen. These cracks started at the bottom surface of a specimen and were essentially perpendicular to the plane of zero bending strain. The transverse load acting on the specimen when the first visible crack was detected is shown on Fig. 5.7 by the notation P_c. As the load was increased, additional concrete cracks appeared, and the existing cracks widened and extended upward. Further load increases caused the prestressing strand to elongate significantly and eventually the top surface of the concrete experienced a compression failure. The ultimate load on this specimen is noted as P_u in Fig. 5.7. For this specimen, strand-slip did not happen and a shear failure did not occur. Therefore, the failure mechanism for Specimen No. 10-6.0DU-9 was classified as a flexural failure. If a flexural failure occurred, the transverse load position was moved closer to the nearest support for the next test on the opposite end of the same specimen or on a new and essentially identical specimen.

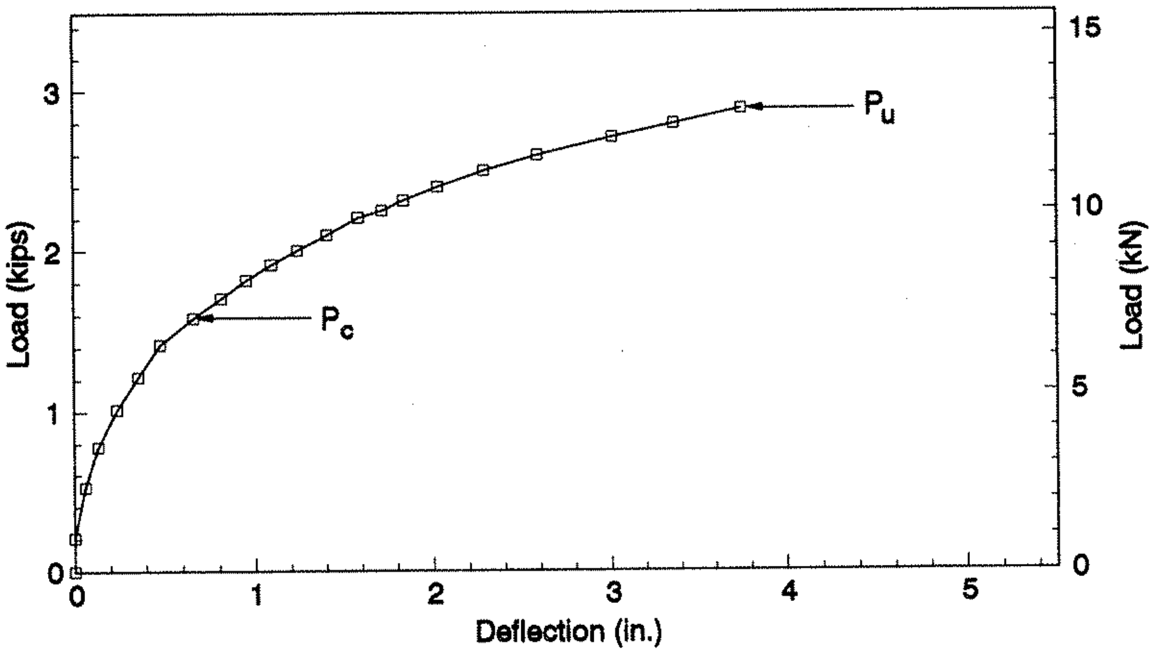
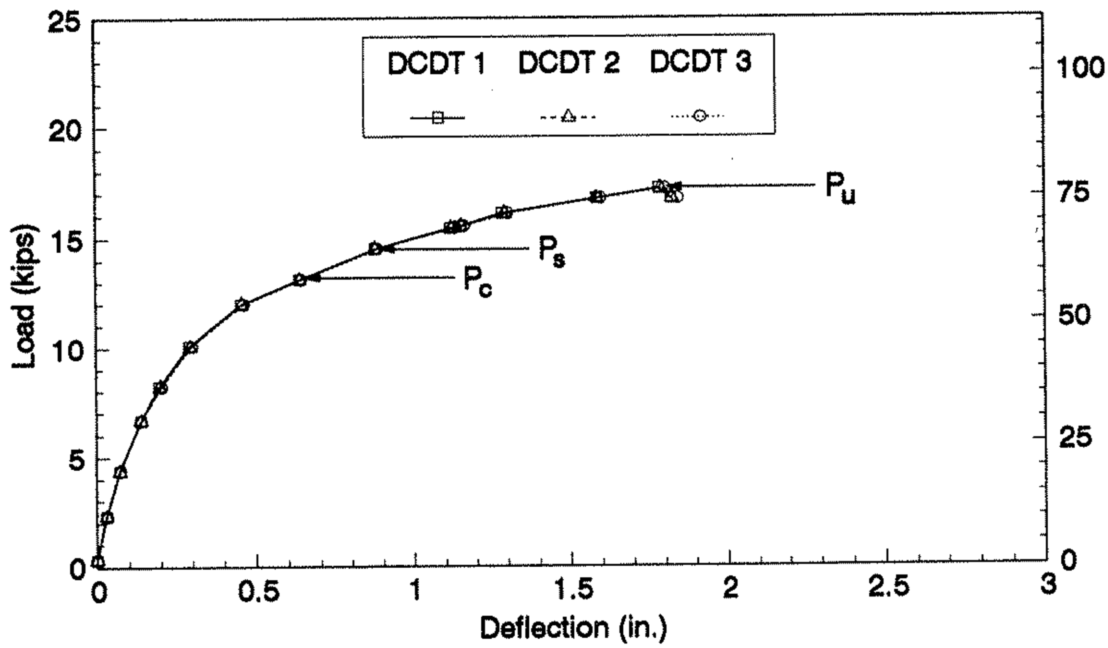


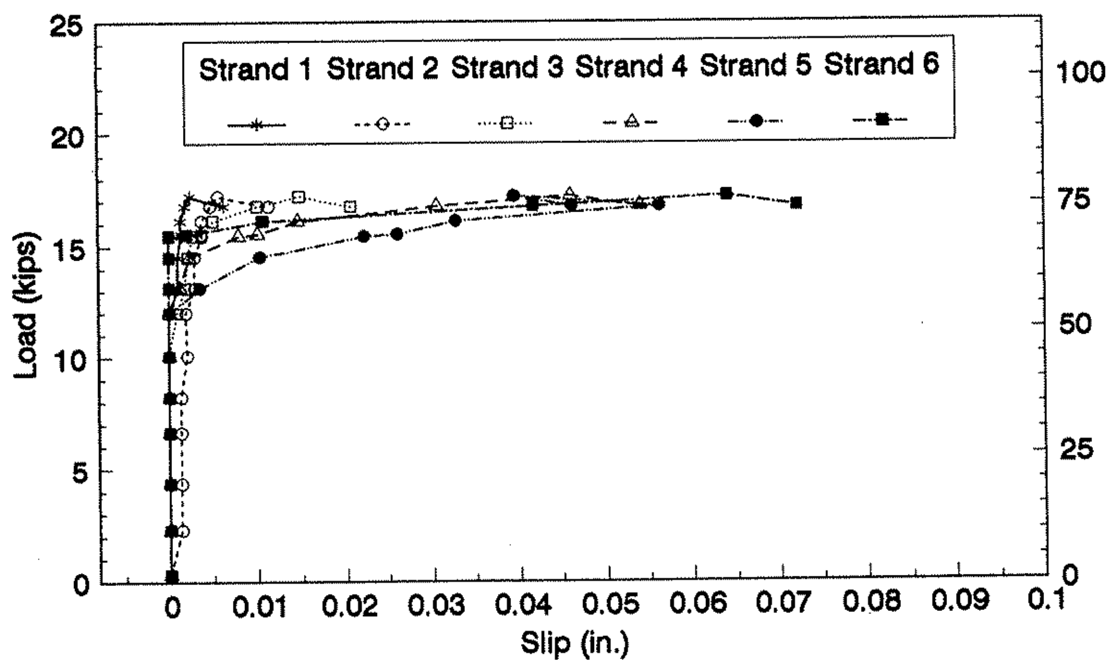
Figure 5.7. Load versus load-point deflection for Specimen No. 10-6.0DU-9 with the load at 54 in. from End E (flexural failure mode)

If the flexural and shear strengths of a specimen were adequate, a bond failure of a prestressing strand occurred when the distance from the near end of a specimen to the transverse load position was shorter than the strand development length. When a strand-slip exceeded 0.01 in., a bond failure was considered to have occurred. Figure 5.8. shows the load versus load-point deflection at 46 in. from End D and the load versus strand-slip relationships at End D for Specimen 15-6.0DU-3. As shown in Fig. 5.8a, the first strand-slip load P_s occurred before the load P_u was reached. In some incidents, a specimen was able to resist additional load after the occurrence of the first bond failure. The bond failure behavior of a multiple-strand specimen was more complex than that of a single-strand specimen. For a 36-in. wide multiple-strand specimen, not all of the strands would slip simultaneously. After the first strand-slip occurred, additional load would cause either additional strand slippages or failure of the specimen before all of the strands experienced slip. Conservatively, the development length of the strands in a multiple-strand specimen was defined as the strand embedment length required to prevent a single strand from experiencing a bond failure before the nominal flexural strength of the specimen was achieved. Figure 5.8b shows the load versus strand-slip behavior for End D of Specimen No. 15-6.0-DU-3. This specimen experienced multiple strand-slips. Since a flexural or shear failure of the specimen did not occur, the failure mechanism for Specimen No. 15-6.0DU-3 was classified as a bond failure.

In some cases, when the transverse load was positioned close to a support for a specimen, the load needed to cause either a flexural or a bond failure of the specimen was higher than the load required to induce a shear failure of the specimen. If only a shear failure occurred in a specimen, strand development length information could not be directly obtained from that specimen, since the



(a)



(b)

Figure 5.8. Load versus displacement relationships for Specimen No. 15-6.0DU-3 with the load at 46 in. from End D (bond failure mode): (a) load versus load-point deflection; (b) load versus strand-slip at End D

shear and strand bond strengths are not related to each other. Unlike a flexural or a bond failure, a shear failure is classified as a brittle failure. For those specimens that experienced a shear failure, the shear cracks developed as an extension of existing flexural-tension cracks that were located near a support. At the ultimate load, one of these shear cracks suddenly extended and arched towards the compression face of the specimen causing a total collapse of the specimen before a flexural or bond failure could occur. Figure 5.9. shows the load versus load-point deflection relationship at 21 in. from End E for Specimen No. 17-6.0DC-12 which failed in shear.

When a particular specimen experienced a failure that involved a combination of two of the primary failure modes, the interpretation of the test results became complex. When a bond failiure occurred in combination with either a flexural or a shear failiure, a strand-slip may have occurred before, at essentially the same time, or after the ultimate load was applied to the specimen. Even though the interaction of the primary failure modes affected the behavior of the test specimens, qualitative observations were made regarding the strand development length. If a combined bond and flexural failure occurred in a specimen, the distance from the transverse load position to the near end of the specimen was essentially equal to the strand development length. Figure 5.10a shows the load versus load-point deflection relationship at 23 in. from End E for Specimen No. 12-6.0DC-3. Figure 5.10b shows the load versus strand-slip relationships at End E. This specimen experienced a combined bond and flexural failure. Figure 5.10a shows that the loads P_u and P_s were essentially equal. If a combined flexural and shear failure occurred in a specimen, the distance from the transverse load to the near end of the specimen was greater than the strand development length. If the specimen failure involved both the shear and bond mechanisms, the distance from the transverse load to the near end of the specimen was less than the strand development length.

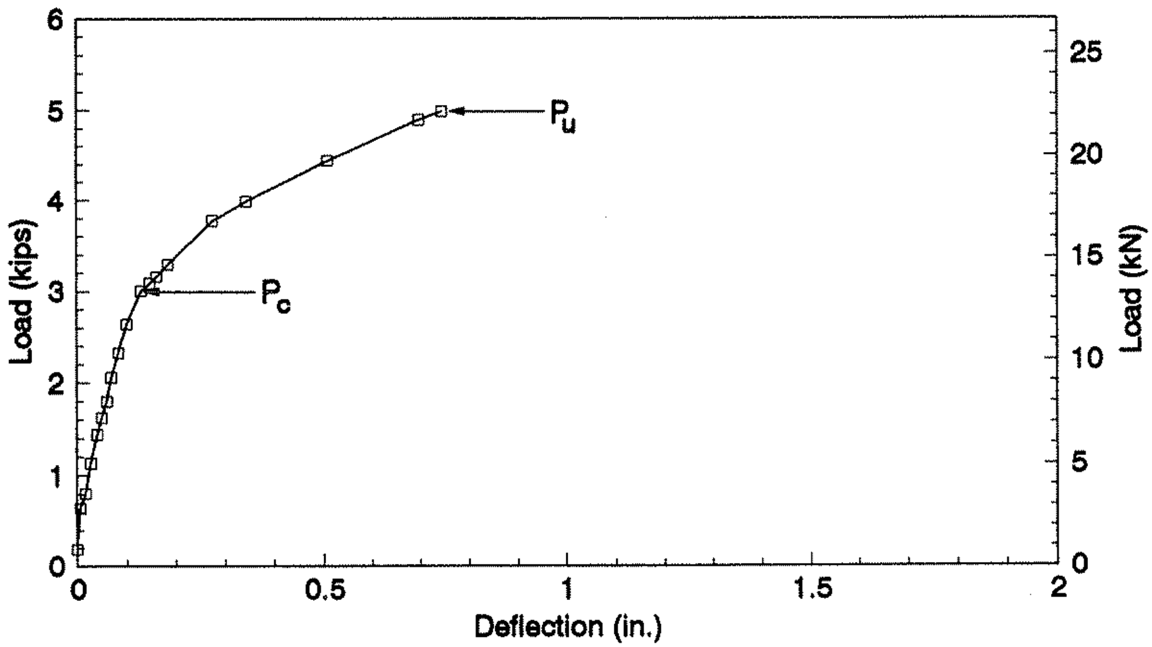


Figure 5.9. Load versus load-point deflection for Specimen No. 17-6.0DC-12 with the load at 21 in. from End E (shear failure mode)

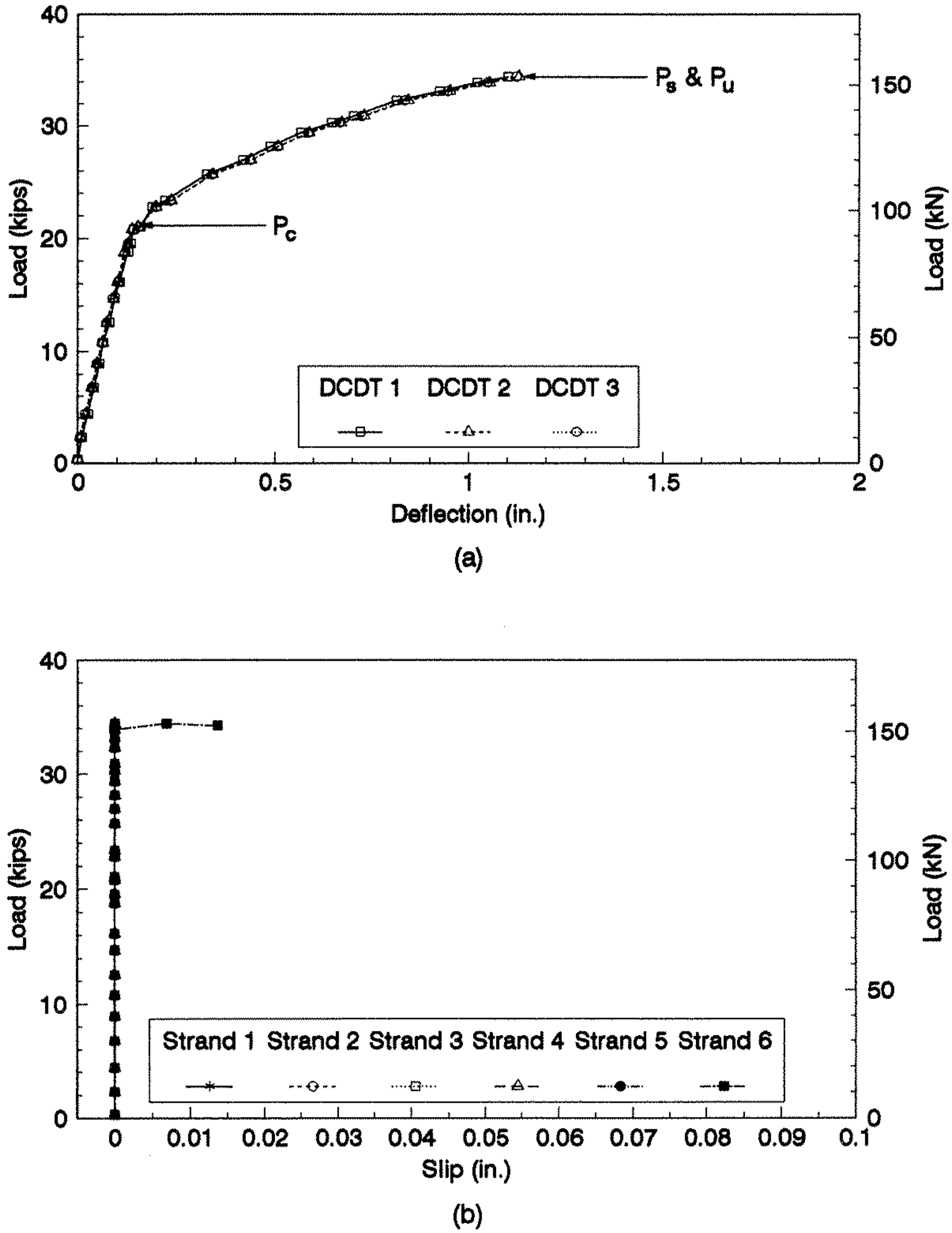


Figure 5.10. Load versus displacement relationships for Specimen No. 12-6.0DC-3 with the load at 23 in. from End E (bond and flexural failure modes): (a) load versus load-point deflection; (b) load versus strand-slip at End E

The concrete crack patterns that formed in most of the D-type specimens were recorded during the strand development length tests. Figure 5.11 shows the concrete cracks in the top surface and two longitudinal edges of Specimen No. 12-6.0DC-1 that developed during each of the two tests that were conducted on this specimen. For the first test, the transverse load was positioned 22 in. from End B. For the second test, the transverse was located 24 in. from End A. The order in which the vertical concrete cracks formed and their relative location along the longitudinal edges of the specimen for each test is noted by the numbers shown below the cracks. The plan view of the specimens reveals that longitudinal concrete splitting occurred in both tests. A more detailed description of the failure for each test on this specimen is represented by the notation given within the parenthesis in Fig. 5.11. The specimen failure at End B started with a bond failure for Strand Nos. 5 and 6. An additional load that was applied after these strand-slips occurred produced a shear failure. This failure sequence was abbreviated by the notation B5, B6; S. For the strand development length test near End A, the specimen failure involved a combination of flexure and shear modes; therefore, the notation FS is shown. The crack patterns and failure sequences for the strand development length tests that were performed on the other D-type specimens are given in Appendix D.

5.5.2. Experimental Results for Strand Development Lengths

The results for the strand development length tests for the 4 and 6-in. wide, single-strand and 36-in. wide, multiple-strand D-type specimens that were prestressed with coated strands are summarized in Tables 5.13, 5.14, and 5.15, respectively. The experimental test results for the comparable specimen containing uncoated strands are given in Tables 5.16, 5.17, and 5.18, respectively. For each specimen listed in the tables, the end of the specimen from which the strand

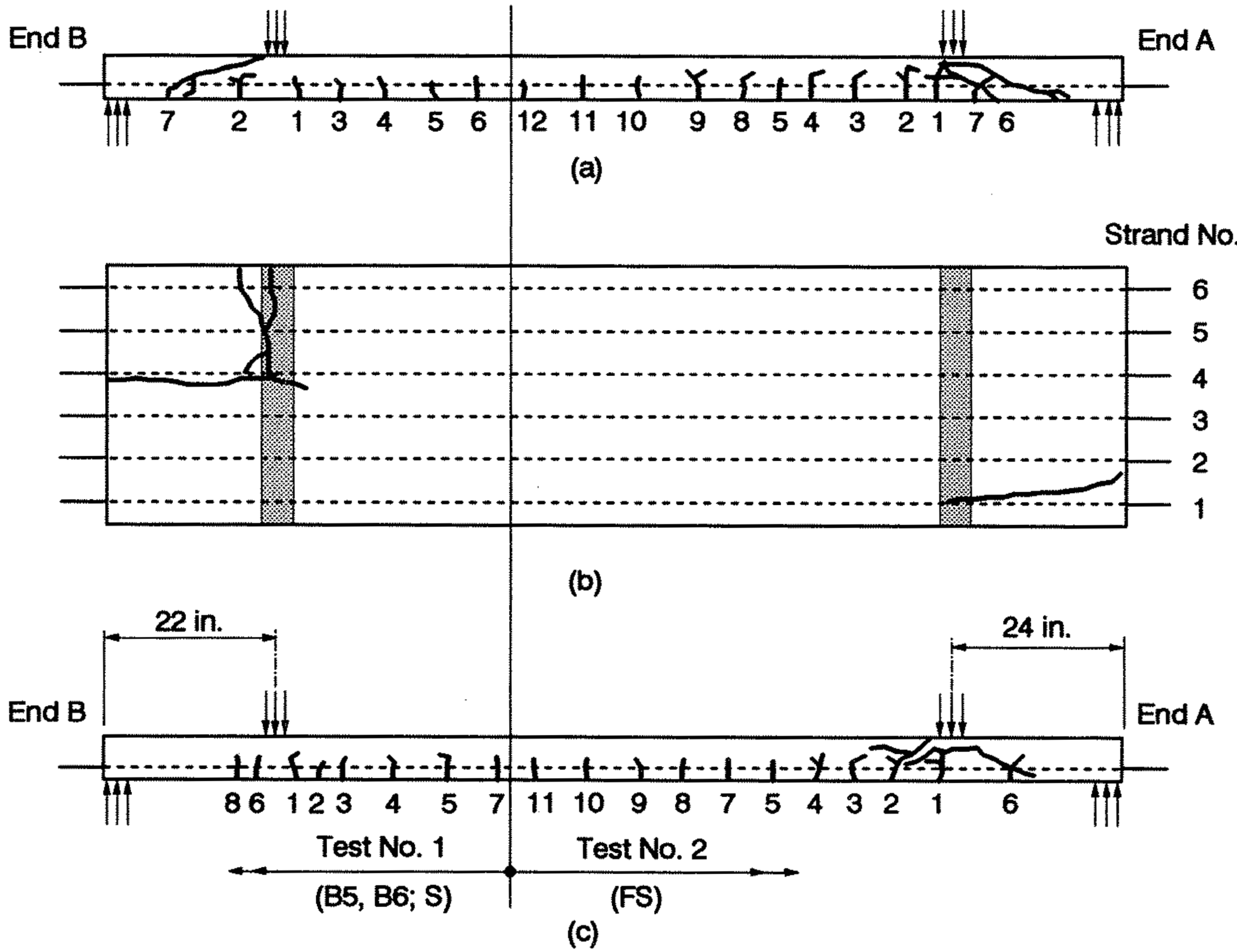


Figure 5.11. Crack patterns for D-type Specimen No. 12-6.0DC-1:
(a) mirrored side view along Strand No. 6; (b) top view;
(c) side view along Strand No. 1

Table 5.13. Strand development length test results for 4-in. wide, coated single-strand specimens

Specimen No.	End	f'_{cd} (psi)	X (in.)	P_c (kips)	P_s (kips)	P_u (kips)	Failure Mode
17-6.0DC-1	B	5130	24	2.863	NA	4.208	FS
	A	5130	26	2.357	NA	3.933	FS
17-6.0DC-2	B	5130	23	2.872	NA	4.404	S
17-6.0DC-3	B	5130	22	3.107	4.626[a]	4.998	B/S
17-6.0DC-4	B	5130	22	2.830	4.192[a]	4.336	B/S

[a]Strand-slip after P_u was reached

Table 5.14. Strand development length test results for 6-in. wide, coated single-strand specimens

Specimen No.	End	f'_{cd} (psi)	X (in.)	P_c (kips)	P_s (kips)	P_u (kips)	Failure Mode
17-6.0DC-9	E	5130	22	2.970	NA	5.242	S
17-6.0DC-10	E	5130	24	2.650	NA	4.678	FS
17-6.0DC-11	E	5130	18	3.774	NA	5.320	S
17-6.0DC-12	E	5130	21	3.083	NA	4.991	S

Table 5.15. Strand development length test results for 36-in. wide, coated multiple-strand specimens

Specimen No.	End	f'_{cd} (psi)	X (in.)	P_c (kips)	P_s (kips)	P_u (kips)	Failure Mode
8-6.0DC-1	B	5120	28	15.10	NA	26.82	F
	A	5120	26	17.08	NA	29.64	F
8-6.0DC-3	D	5120	21.5	22.55	28.40[a,i] 21.16[g,i]	29.16	B/S
	E	5120	24	17.28	27.08[a,i] 26.38[f,i] 25.90[c,i] 25.20[d,i] 25.10[b,i] 24.47[e,i]	27.73	B
9-6.0DC-1	A	5710	26	16.00	27.03[h,i]	27.07	S/B
	B	5710	24	17.13	26.10[c] 27.39[b] 27.69[a]	28.02	B/S
9-6.0DC-3	E	5710	25	16.22	NA	29.37	FS
12-6.0DC-1	B	6140	22	21.83	29.08[e] 30.33[f]	30.63	B/S
	A	6140	24	19.38	NA	33.55	FS
12-6.0DC-3	E	6140	23	21.02	34.26[f,i]	34.45	B/F
	D	6140	22	21.06	31.46[e,i] 30.91[f,i]	31.75	B/S

[a]Strand No. 1 slipped
[b]Strand No. 2 slipped
[c]Strand No. 3 slipped
[d]Strand No. 4 slipped
[e]Strand No. 5 slipped
[f]Strand No. 6 slipped
[g]Strand Nos. 2-6 slipped
[h]Strand No. 2 slip = 0.006 in.
[i]Strand-slip after P_u was reached

Table 5.15. Strand development length test results for 36-in. wide, coated multiple-strand specimens (continued)

Specimen No.	End	f'_{cd} (psi)	X (in.)	P_c (kips)	P_s (kips)	P_u (kips)	Failure Mode
14-6.0DC-1	A	5230	20	20.23	32.26[a,i] 31.37[c,i] 30.35[b,i] 29.91[d,i] 29.07[f,i]	32.49	B
	B	5230	24	17.59	30.10[f,i] 24.26[e,i]	30.79	B/S
14-6.0DC-3	E	5230	22	17.08	NA	30.54	S
	D	5230	24	17.71	21.59[c] 26.19[f,i]	27.11	B/S

[a]Strand No. 1 slipped
[b]Strand No. 2 slipped
[c]Strand No. 3 slipped
[d]Strand No. 4 slipped
[e]Strand No. 5 slipped
[f]Strand No. 6 slipped
[g]Strand Nos. 1-4 & 6 slipped
[i]Strand-slip after P_u was reached

Table 5.16. Strand development length test results for 4-in. wide, uncoated single-strand specimens

Specimen No.	End	f'_{cd} (psi)	X (in.)	P_c (kips)	P_s (kips)	P_u (kips)	Failure Mode
10-6.0DU-1	A	5350	70.5	1.417	NA	2.502	F
10-6.0DU-2	A	5350	65	1.341	1.916	2.595	B[a]
10-6.0DU-3	B	5350	56	1.317	1.408	1.700	B
	A	5350	60	1.655	2.145	2.624	B
10-6.0DU-4	B	5350	54	1.319	1.314	1.963	B
11-6.0DU-1	B	6150	60	1.723	NA	2.464	F
11-6.0DU-2	B	6150	55	1.547	NA	2.504	F
11-6.0DU-3	B	6150	50	1.657	2.488[b]	2.519	B/F
	A	6150	52	1.645	NA	2.637	F
11-6.0DU-4	B	6150	51	1.564	NA	2.538	F

[a]Strand-slip at P_s = 1.776 kips at End B
[b]Strand-slip after P_u was reached

Table 5.17. Strand development length test results for 6-in. wide, uncoated single-strand specimens

Specimen No.	End	f'_{cd} (psi)	X (in.)	P_c (kips)	P_s (kips)	P_u (kips)	Failure Mode
10-6.0DU-9	D	5350	50	1.704	1.714	2.290	B
	E	5350	54	1.585	NA	2.883	F
10-6.0DU-10	E	5350	54	1.736	2.720[b]	2.795	B/F
10-6.0DU-11	E	5350	44	1.261	1.288	2.265	B
	D	5350	46	1.195	1.358	2.139	B
10-6.0DU-12	E	5350	42	1.651	1.683	2.404	B[a]
11-6.0DU-9	D	6150	50	1.736	NA	2.802	F
11-6.0DU-10	D	6150	45	1.867	NA	3.101	F
11-6.0DU-11	D	6150	40	2.020	3.356[b]	3.402	B
	E	6150	40	2.025	3.102	3.215	B
11-6.0DU-12	D	6150	42	1.900	3.017	3.075	B
	E	6150	44	1.826	2.854	3.052	B

[a]Strand-slip at P_s = 2.041 kips at End D
[b]Strand-slip after P_u was reached

Table 5.18. Strand development length test results for 36-in wide, uncoated multiple-strand specimens

Specimen No.	End	f'_{cd} (psi)	X (in.)	P_c (kips)	P_s (kips)	P_u (kips)	Failure Mode
6-6.0DU-1	A	2920	50	NR	NA	14.98	F
6-6.0DU-3	E	2920	30	NR	10.26[f] 11.32[a]	11.33	B
	D	2920	40	NR	10.93[d] 11.91[i] 12.37[b] 8.54[a,l]	12.86	B
7-6.0DU-3	D	4890	45	NR	14.39[a] 15.42[e] 15.87[b]	16.70	B[j]
7-6.0DU-1	B	4890	45	11.01	15.07[a,l]	15.22	B[k]
	A	4890	42	12.07	10.29[a] 12.27[b] 15.26[h] 14.58[g,l]	15.54	B
15-6.0DU-1	A	5440	42	12.08	18.03[a]	18.99	B[k]
	B	5440	44	11.69	12.04[e] 12.85[a] 15.70[f] 16.43[d] 15.44[b,l]	17.35	B
15-6.0DU-3	E	5440	48	11.99	NA	18.97	F
	D	5440	46	12.01	14.53[e] 15.55[d] 16.12[f] 16.79[c] 16.78[b,l] 16.52[a,l]	17.22	B

[a]Strand No. 1 slipped
[b]Strand No. 2 slipped
[c]Strand No. 3 slipped
[d]Strand No. 4 slipped
[e]Strand No. 5 slipped
[f]Strand No. 6 slipped
[g]Strand Nos. 3 & 5 slipped
[h]Strand Nos. 4 & 6 slipped
[i]Strand Nos. 3, 5 & 6 slipped
[j]Strand No. 1 slipped at P_s = 13.85 kips at End E
[k]Test stopped after first strand-slip when a flexural failure was imminent
[l]Strand-slip after P_u was reached

Table 5.18. Strand development length test results for 36-in. wide, uncoated multiple-strand specimens (continued)

Specimen No.	End	f'_{cd} (psi)	X (in.)	P_c (kips)	P_s (kips)	P_u (kips)	Failure Mode
16-6.0DU-1	B	5870	48	11.49	14.49[f] 14.99[e] 16.20[a] 16.60[d] 16.61[c,g] 16.28[b,g]	16.90	B
	A	5870	49	11.32	NA	19.08	F
16-6.0DU-3	D	5870	45	11.36	14.04[c] 14.54[a] 14.70[e] 14.97[b] 15.80[d] 16.45[f,g]	16.52	B
	E	5870	50	10.95	NA	17.35	F

[a]Strand No. 1 slipped
[b]Strand No. 2 slipped
[c]Strand No. 3 slipped
[d]Strand No. 4 slipped
[e]Strand No. 5 slipped
[f]Strand No. 6 slipped
[g]Strand-slip after P_u was reached

development length was measured is noted by the letter (A, B, C, D, or E) that corresponds with the single, I-shaped, steel header designation for the prestressing frame (see Fig. 3.1). The concrete compressive strength f'_{cd} was measured when the particular strand development length test was conducted. The distance X listed in the tables and shown in Fig. 3.7 refers to the dimension from the transverse load position to the end of the specimen. The loads P_c, P_s, and P_u, corresponding to the transverse loads acting on a specimen when the first concrete crack was detected, when a strand-slip reached 0.10 in., and when the ultimate strength was obtained, respectively. For some of the specimens, the notation NR and NA shown in these tables signifies that this specific data was not recorded and not applicable, respectively. The last column in these tables specifies the failure mode for a particular specimen.

When a particular specimen experienced a B, B/S, S/B, or B/F failure mode, the first strand-slip load P_s listed in Tables 5.13-5.18 for that specimen corresponds to the applied load that was present when the first strand-slip occurred. Subsequent P_s loads for the same specimen are listed in order of occurrence and correspond to the loads that were acting when the bond strength for the other strands were exceeded. The P_s loads given in Table 5.15 for Specimen No. 8-6.0DC-3 for End E all occurred after the ultimate load P_u was reached. However, the first strand-slip load was almost equal to the load P_u. The order of the bond failures for each strand in this specimen are indicated by the footnotes given in the table. End D of Specimen No. 15-6.0DU-3 experienced strand-slips before and after the ultimate load was reached.

For some of the specimens listed in Tables 5.13-5.18, not all of the strand-slip loads are given. The omitted strand-slip loads occurred after the ultimate load P_u had been reached and when the applied load, which could still be resisted by the specimen, was significantly smaller than the load P_u.

When a specimen failure involved a combination of bond and shear or bond and flexural failure modes, the strand-slips that occurred well after the ultimate load P_u was obtained were probably secondary failures that resulted from the specimen damage caused by a shear or flexural failure. Therefore, these P_s loads are not accurate indications of the bond strength, so they have not been listed.

Development lengths for 3/8-in. diameter, seven-wire, 270-ksi, low-relaxation, prestressing strands can be interpreted from the experimental results that are presented in Tables 5.13 through 5.15 for coated strands and Table 5.16 through 5.18 for uncoated strands. For the specimens that had the same geometric configuration and a specific strand coating, an observation of the failure mode that occurred with each transverse load position produced the ranges and the specific strand development lengths shown in Table 5.19. A comparison of the coated-strand development lengths listed in the table for the 4-in. wide and 6-in. wide, single-strand specimens and for the 6-in. wide, single-strand and 36-in. wide, multiple-strand specimens revealed that the development length for the coated strands does not appear to be affected by the amount of concrete edge cover used in the test specimens and by the spacing of the strands present in the multiple-strand specimens, respectively. A similar comparison of the strand development lengths for the uncoated strand specimens revealed that the development length for the uncoated strands appears to be influenced by the amount of concrete edge cover. The uncoated strand development length does not appear to be affected by the strand spacing. A closer examination of strand development lengths is presented in Section 5.5.3.

Table 5.19. Experimental development lengths for 3/8-in. diameter, seven-wire, 270 ksi, low-relaxation, prestressing strands

Specimen Size (in.)		Coated Strands			Uncoated Strands		
b	h	Cast No.	L_d (in.)	f'_{cd} (psi)	Cast No.	L_d (in.)	f'_{cd} (psi)
4	6	17	22-24	5130	10	65-70	5350
		-	-	-	11	51	6150
6	6	17	<24	5130	10	54	5350
		-	-	-	11	45	6150
36	6	8	26	5120	6	40-50	2920
		9	25	5710	7	45	4890
		12	24	6140	15	48	5440
		14	>24	5230	16	49	5870
Average		-	25[a]	5410	-	50[b,c]	5600[c]

[a]Used upper limit for Cast No. 17 and lower bound for Cast No. 14
[b]Used upper limit for Cast No. 10
[c]Cast No. 6 was omitted due to low concrete strength

5.5.3. Nondimensional Analysis of Strand Development Lengths

As discussed in Section 5.5.1, three primary (failure modes F, B, and S) and five combination (failure modes F/B, B/F, FS, B/S, and S/B) failure mechanisms were possible for a test specimen. For the purposes of a nondimensional analysis of the strand development lengths, the specimen failures that involved a combination of two primary modes have been reclassified on the basis of the initial primary failure mode. Therefore, if a particular specimen experienced an initial bond failure which was immediately followed by a flexural failure, the mode of failure has been reclassified as a bond failure. Conversely, if a flexural failure induced a subsequent bond failure, the failure mechanism was reclassified as a flexural failure. Similarly, for specimen failures involving a combination of flexure and shear and bond and shear, the failure mode that occurred first has been selected as the initial mechanism which initiated the failure of the particular specimen. If a flexural and a bond or shear and a bond failure appeared to occur simultaneously, the bond failure was selected as the initial mechanism. When a flexural and a shear failure occurred essentially simultaneously, the flexural failure was considered to be the initial failure mechanism.

The data used for the nondimensional analysis of the strand development length for the coated strands in the 4 and 6-in. wide, single-strand and 36-in. wide, multiple-strand specimens is presented in Tables 5.20, 5.21 and 5.22, respectively. The same type of information for the uncoated strands in presented in Tables 5.23, 5.24, and 5.25, respectively. For each of these tables, the concrete cracking moment M_c, strand-slip moment M_s, and ultimate moment M_u, are the moments induced by the transverse load at the load-point cross section when the first visible concrete crack was detected, when any strand experience an end-slip of 0.01 in., and when the ultimate load was applied, respectively. For specimens that had an initial flexural or shear failure, the critical moment M_{cr} is the

Table 5.20. Relationships for the induced and nominal bending moments and shear forces and load position for the 4-in. wide, coated single-strand specimens

Specimen No.	End	Failure Modes		X (in.)	M_c (k-in.)	M_s (k-in.)	M_u (k-in.)	V_u (kip)	M_n (k-in.)	V_n (kip)	L_d[a] (in.)	M_{cr}/M_n	X/L_d	V_u/V_n	X/L
		Total	Initial												
17-6.0DC-1	B	FS	F	24	52.87	NA	77.71	3.532	76.28	3.029	50.1	1.019	0.479	1.166	0.170
	A	FS	F	26	46.66	NA	77.86	3.244	76.28	2.857	50.1	1.021	0.519	1.135	0.184
17-6.0DC-2	B	S	S	23	51.07	NA	78.31	3.729	76.28	3.031	50.1	1.027	0.459	1.230	0.163
17-6.0DC-3	B	B/S	B	22	53.07	79.01	85.37	4.268	76.28	3.234	50.1	1.036	0.439	1.320	0.156
17-6.0DC-4	B	B/S	B	22	48.34	71.60	74.06	3.703	76.28	3.234	50.1	0.939	0.439	1.145	0.156

[a]From AASHTO Specification [1]

Table 5.21. Relationships for the induced and nominal bending moments and shear forces and load position for the 6-in. wide, coated single-strand specimens

Specimen No.	End	Failure Modes		X (in.)	M_c (k-in.)	M_s (k-in.)	M_u (k-in.)	V_u (kip)	M_n (k-in.)	V_n (kip)	L_d[a] (in.)	M_{cr}/M_n	X/L_d	V_u/V_n	X/L
		Total	Initial												
17-6.0DC-9	E	S	S	22	50.73	NA	89.53	4.477	86.19	4.034	50.1	1.039	0.439	1.110	0.156
17-6.0DC-10	E	FS	F	24	48.94	NA	86.39	3.927	86.19	3.801	50.1	1.002	0.479	1.033	0.170
17-6.0DC-11	E	S	S	18	53.33	NA	75.18	4.699	86.19	4.669	50.1	0.872	0.359	1.006	0.128
17-6.0DC-12	E	S	S	21	50.45	NA	81.68	4.299	86.19	4.169	50.1	0.948	0.419	1.031	0.149

[a]From AASHTO Specification [1]

Table 5.22. Relationships for the induced and nominal bending moments and shear forces and load positions for the 36-in. wide,coated multiple-strand specimens

Specimen No.	End	Failure Modes		X (in.)	M_c (k-in.)	M_s (k-in.)	M_u (k-in.)	V_u (kip)	M_n (k-in.)	V_n (kip)	L_d[a] (in.)	M_{cr}/M_n	X/L_d	V_u/V_n	X/L
		Total	Initial												
8-6.0DC-1	B	F	F	28	318.0	NA	565.0	21.73	510.9	21.02	50.2	1.106	0.558	1.034	0.199
	A	F	F	26	338.0	NA	586.7	24.45	510.9	21.61	50.2	1.148	0.518	1.131	0.184
8-6.0DC-3	D	B/S	B	21.5	377.2	474.9	487.6	25.00	510.9	24.58	50.2	0.930	0.428	1.117	0.152
	E	B	B	24	319.0	500.0	512.1	23.28	510.9	22.79	50.2	0.979	0.478	1.022	0.170
9-60DC-1	A	S/B	S	26	316.7	NA	535.8	22.33	523.0	22.26	50.2	1.024	0.518	1.003	0.184
	B	B/S	B	24	316.3	482.0	517.4	23.52	523.0	23.45	50.2	0.922	0.478	1.003	0.170
9-6.0DC-3	E	FS	F	25	310.4	NA	562.1	24.44	523.0	22.83	50.2	1.075	0.498	1.071	0.177
12-6.0DC-1	B	B/S	B	22	372.8	496.6	523.2	26.16	528.9	25.35	50.3	0.939	0.437	1.032	0.156
	A	FS	S	24	357.9	NA	619.6	28.16	528.9	23.91	50.3	1.171	0.477	1.178	0.170
12-6.0DC-3	E	B/F	B	23	373.8	609.1	612.5	29.17	528.9	24.59	50.3	1.152	0.457	1.186	0.163
	D	B/S	B	22	359.7	537.4	542.2	27.11	528.9	25.35	50.3	1.016	0.437	1.069	0.156
14-6.0DC-1	A	B	B	20	316.4	504.4	508.0	28.22	513.6	26.02	50.2	0.983	0.398	1.085	0.142
	B	B/S	B	24	324.8	555.9	568.6	25.85	513.6	22.92	50.2	1.082	0.478	1.128	0.170
14-6.0DC-3	E	S	S	22	291.6	NA	521.6	26.08	513.6	24.32	50.2	1.016	0.438	1.072	0.156
	D	B/S	B	24	327.0	398.6	500.6	22.75	513.6	22.92	50.2	0.942	0.478	0.993	0.170

[a]From AASHTO Specification [1]

Table 5.23. Relationships for the induced and nominal bending moments and shear forces and load positions for the 4-in. wide, uncoated single-strand specimens

Specimen No.	End	Failure Modes		X (in.)	M_c (k-in.)	M_s (k-in.)	M_u (k-in.)	V_u (kip)	M_n (k-in.)	V_n (kip)	L_d[a] (in.)	M_{cr}/M_n	X/L_d	V_u/V_n	X/L
		Total	Initial												
10-6.0DU-1	A	F	F	70.5	48.53	NA	85.69	1.251	77.13	2.387	50.2	1.111	1.404	0.524	0.500
10-6.0DU-2	A	B	B	65	45.63	65.20	88.31	1.402	77.13	2.387	50.2	0.845	1.295	0.587	0.461
10-6.0DU-3	B	B	B	56	43.09	46.06	55.62	1.030	77.13	2.387	50.2	0.597	1.116	0.432	0.397
	A	B	B	60	55.35	71.74	87.76	1.513	77.13	2.387	50.2	0.930	1.195	0.634	0.426
10-6.0DU-4	B	B	B	54	42.55	42.39	63.33	1.218	77.13	2.428	50.2	0.550	1.076	0.510	0.383
11-6.0DU-1	B	F	F	60	57.63	NA	82.41	1.421	79.97	2.560	50.2	1.044	1.195	0.555	0.426
11-6.0DU-2	B	F	F	55	50.27	NA	81.37	1.535	79.97	2.560	50.2	1.031	1.096	0.610	0.390
11-6.0DU-3	B	B/F	B	50	51.67	77.58	78.59	1.636	79.97	2.560	50.2	0.983	0.996	0.639	0.355
	A	F	F	52	52.23	NA	83.73	1.675	79.97	2.560	50.2	1.061	1.036	0.654	0.369
11-6.0DU-4	B	F	F	51	49.23	NA	79.88	1.630	79.97	2.560	50.2	1.012	1.016	0.637	0.362

[a]From AASHTO Specification [1]

Table 5.24. Relationships for the induced and nominal bending moments and shear forces and load positions for the 6-in. wide, uncoated single-strand specimens

Specimen No.	End	Failure Modes		X (in.)	M_c (k-in.)	M_s (k-in.)	M_u (k-in.)	V_u (kip)	M_n (k-in.)	V_n (kip)	L_d[a] (in.)	M_{cr}/M_n	X/L_d	V_u/V_n	X/L
		Total	Initial												
10-6.0DU-9	D	B	B	50	53.13	53.45	71.41	1.488	87.15	3.581	50.1	0.613	0.998	0.416	0.355
	E	F	F	54	51.14	NA	93.01	1.789	87.15	3.581	50.1	1.067	1.078	0.500	0.383
10-6.0DU-10	E	B/F	B	54	56.01	887.75	90.17	1.734	87.15	3.581	50.1	1.007	1.078	0.484	0.383
10-6.0DU-11	E	B	B	44	36.73	37.51	65.97	1.571	87.15	3.581	50.1	0.430	0.878	0.439	0.312
	D	B	B	46	35.69	40.56	63.89	1452	87.15	3.581	50.1	0.465	0.918	0.405	0.326
10-6.0DU-12	E	B	B	42	46.76	47.66	68.08	1.702	87.15	3.581	50.1	0.547	0.838	0.475	0.298
11-6.0UD-9	D	F	F	50	54.13	NA	87.37	1.820	90.31	3.840	50.2	0.967	0.996	0.474	0.355
11-6.0DU-10	D	F	F	45	55.08	NA	91.49	2.128	90.31	3.840	50.2	1.013	0.896	0.554	0.319
11-6.0DU-11	D	B	B	40	55.47	92.16	93.42	2.458	90.31	3.840	50.2	1.020	0.797	0.640	0.284
	E	B	B	40	55.61	85.18	88.28	2.323	90.31	3.840	50.2	0.943	0.797	0.605	0.284
11-6.0DU-12	D	B	B	42	53.81	85.44	87.09	2.177	90.31	3.840	50.2	0.946	0.837	0.567	0.298
	E	B	B	44	53.18	83.12	88.89	2.116	90.31	3.840	50.2	0.920	0.876	0.551	0.312

[a]From AASHTO Specification [1]

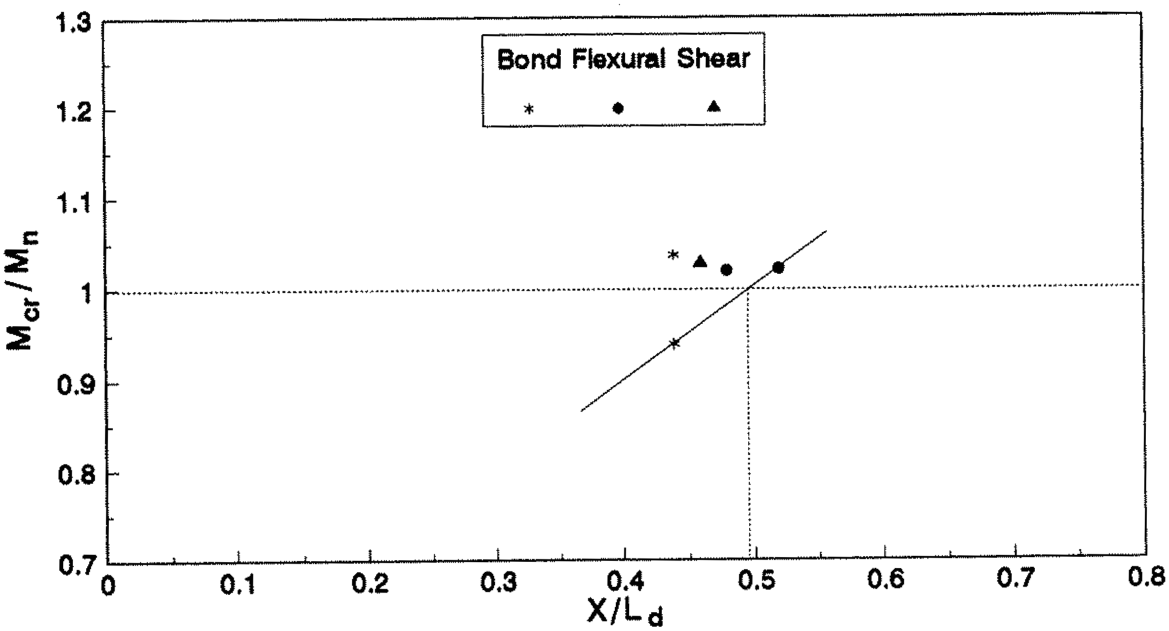
Table 5.25. Relationships for the induced and nominal bending moments and shear forces and load positions for the 36-in. wide, uncoated multiple-strand specimens

Specimen No.	End	Failure Modes		X (in.)	M_c (k-in.)	M_s (k-in.)	M_u (k-in.)	V_u (kip)	M_n (k-in.)	V_n (kip)	L_d[a] (in.)	M_{cr}/M_n	X/L_d	V_u/V_n	X/L
		Total	Initial												
6-6.0DU-1	A	F	F	50	ND	NA	467.0	9.73	438.4	15.87	49.7	1.065	1.006	0.613	0.355
6-6.0DU-3	E	B	B	30	ND	228.6	252.5	9.02	438.4	17.10	49.7	0.521	0.604	0.527	0.213
	D	B	B	40	ND	300.0	353.0	9.29	438.4	15.87	49.7	0.684	0.805	0.585	0.284
7-6.0DU-3	D	B	B	45	ND	424.6	492.7	11.46	505.2	20.54	50.2	0.840	0.896	0.558	0.319
7-6.0DU-1	B	B	B	45	324.8	444.8	449.2	10.45	505.2	20.54	50.2	0.880	0.896	0.508	0.319
	A	B	B	42	341.8	291.3	440.0	11.00	505.2	20.54	50.2	0.577	0.837	0.535	0.298
15-6.0DU-1	A	B	B	42	342.1	510.6	537.9	13.45	518.2	21.67	50.2	0.985	0.837	0.621	0.298
	B	B	B	44	340.5	350.5	505.3	12.03	518.2	21.67	50.2	0.676	0.876	0.555	0.312
15-6.0DU-3	E	F	F	48	366.4	NA	579.5	12.60	518.2	21.67	50.2	1.118	0.956	0.581	0.340
	D	B	B	46	358.7	433.9	514.5	11.69	518.2	21.67	50.2	0.837	0.916	0.540	0.326
16-6.0DU-1	B	B	B	48	350.9	442.7	516.3	11.22	525.3	22.51	50.2	0.842	0.956	0.498	0.340
	A	F	F	49	349.6	NA	589.0	12.53	525.3	22.51	50.2	1.121	0.976	0.557	0.348
16-6.0DU-3	D	B	B	45	335.2	414.3	487.5	11.34	525.3	22.51	50.2	0.788	0.896	0.504	0.319
	E	F	F	50	341.5	NA	541.0	11.27	525.3	22.51	50.2	1.029	0.996	0.501	0.355

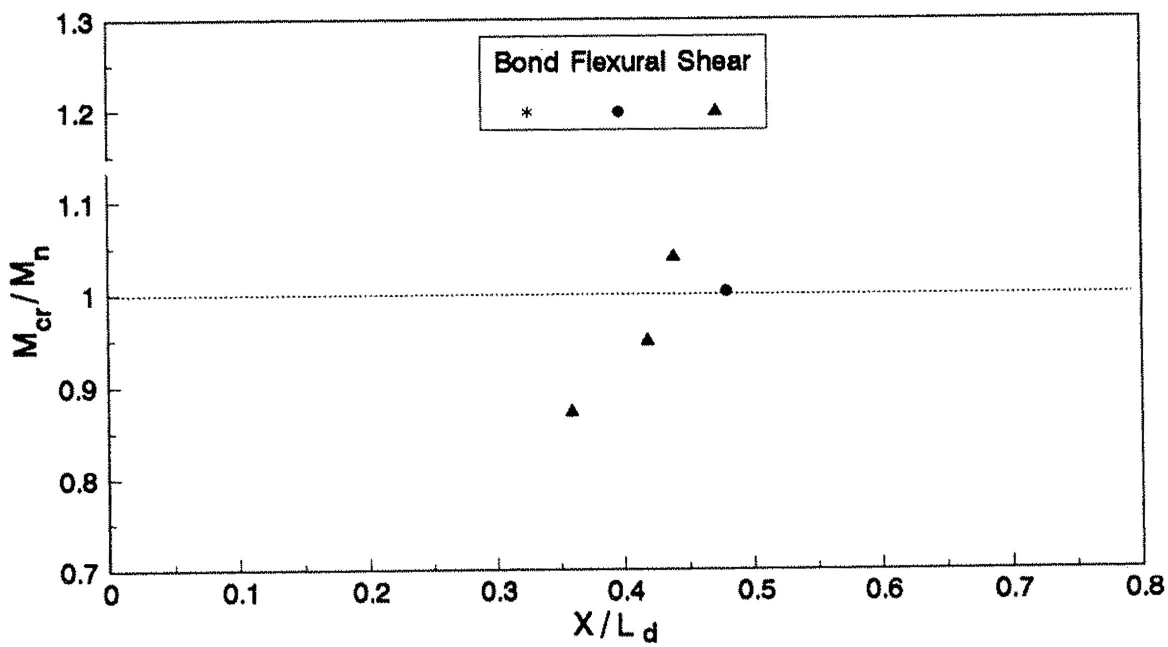
[a]From AASHTO Specification [1]

ultimate moment M_u. For specimens that had an initial bond failure, the critical moment M_{cr} is the strand-slip moment M_s. The ultimate shear force V_u, is the maximum shear force induced by the maximum transverse load that was resisted by a particular specimen. The nominal moment strength M_n and nominal shear strength V_n which were computed by applying the appropriate equations given in Sections 4.5 and 4.6, respectively, were used to nondimensionalize the critical moment M_{cr} and shear force V_u, respectively. The distance X from the near end of a specimen to the location of the transverse load was nondimensionalized by dividing this length by the strand development length L_d obtained from the AASHTO Specification [1] Eq. (9-32) and rewritten here as Eq. (4.27) for the flexural strength study and by the span length L of the specimen for the shear strength study.

The development length for a strand is the length of strand embedment in the concrete that is required to develop the strand prestress f^*_{su}. Since this strand stress is associated with the nominal moment strength of the PC member, the development length of a prestressing strand can be established by investigating the ratio of the induced moment M_{cr} to the moment strength M_n for the specimens. Each nondimensionalized moment (M_{cr}/M_n) and nondimensionalized length (X/L_d) established a data point for a graphical representation of the flexural strength versus strand embedment length relationship. Figures 5.12 and 5.13 show this relationship for the 4, 6, and 36-in. wide specimens that were reinforced with coated and uncoated strands, respectively. To distinguished the three initial failure mechanisms (failure modes F, B, and S) for the specimens, different symbols have been assigned to the corresponding data points shown in these figures. The horizontal line that is drawn through the M_{cr}/M_u-ordinate value that equals unity represents the moment strength condition for which the stress in a strand is equal to f^*_{su} and the nominal moment strength of the specimen is equal to M_n. Figures 5.12 and 5.13 imply that whenever a data point

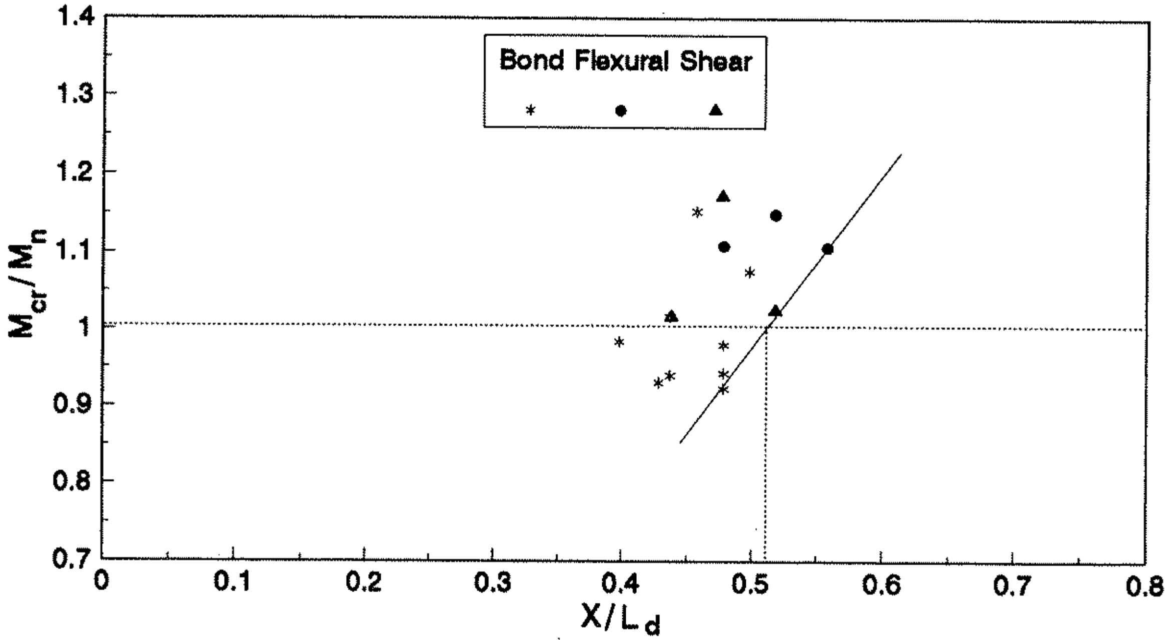


(a)



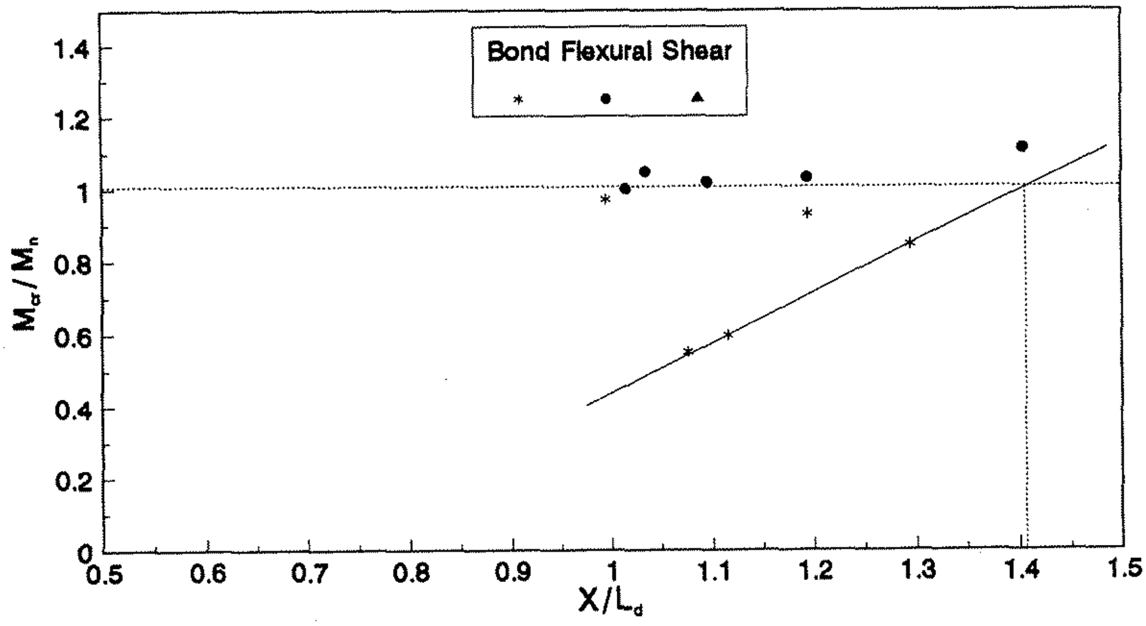
(b)

Figure 5.12. Nondimensionalized relationship of moment strength versus load position for coated strands: (a) 4-in. wide single-strand specimens; (b) 6-in. wide single-strand specimens; (c) 36-in. wide multiple-strand specimens

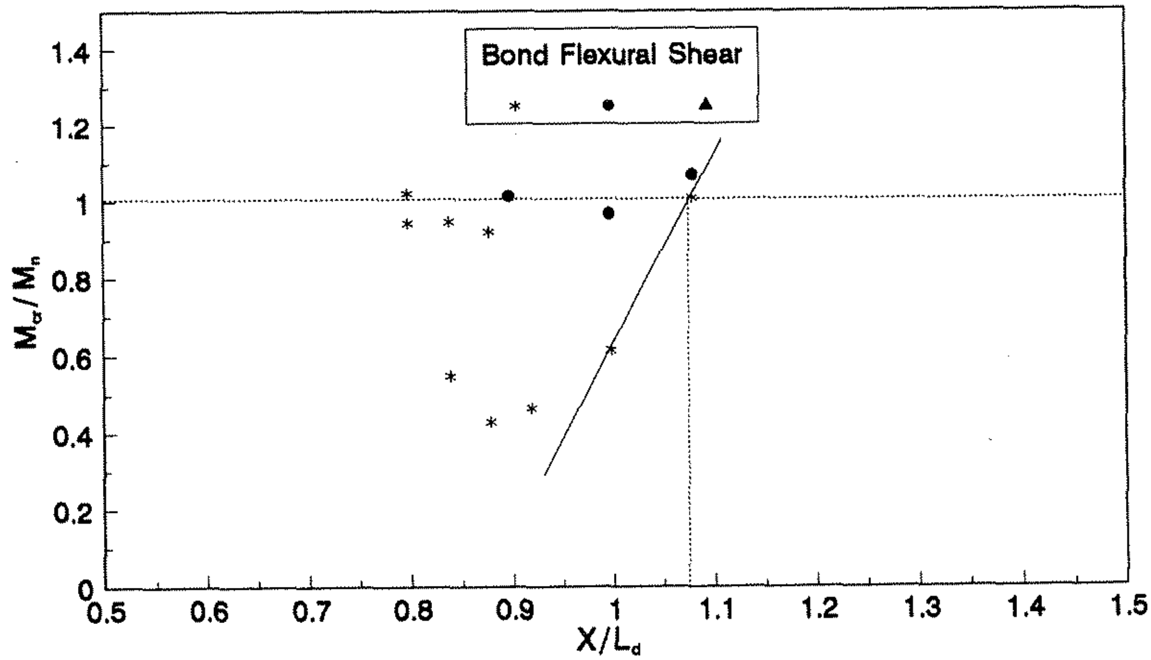


(c)

Figure 5.12. (continued)

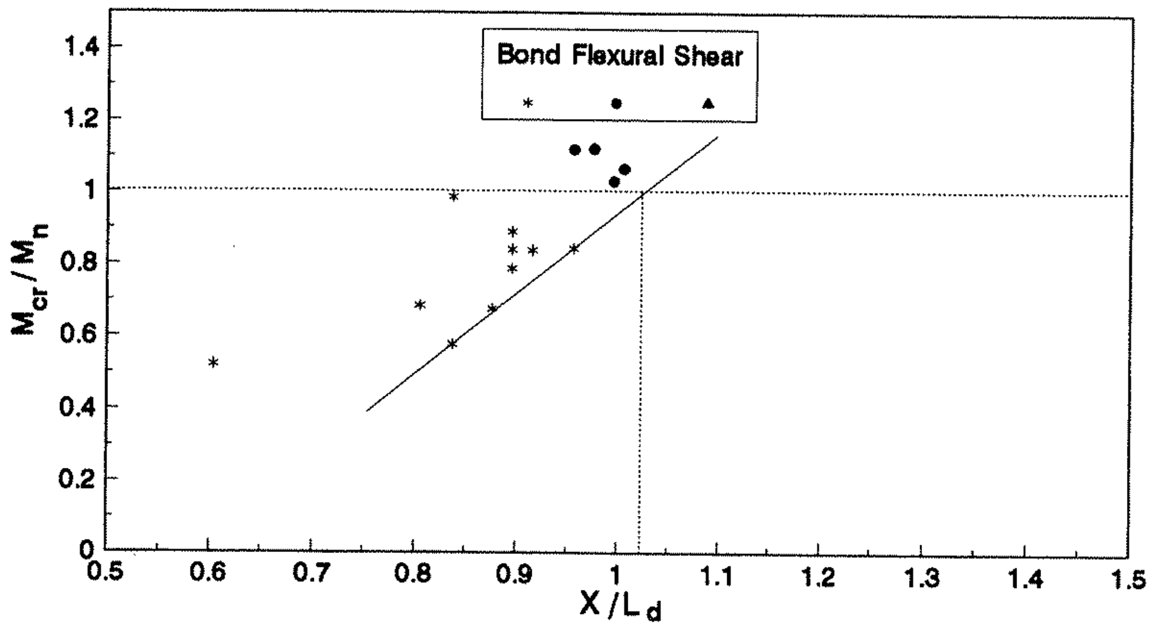


(a)



(b)

Figure 5.13. Nondimensionalized relationship of moment strength versus load position for uncoated strands: (a) 4-in. wide single-strand specimens; (b) 6-in. wide single-strand specimens; (c) 36-in. wide multiple-strand specimens



(c)

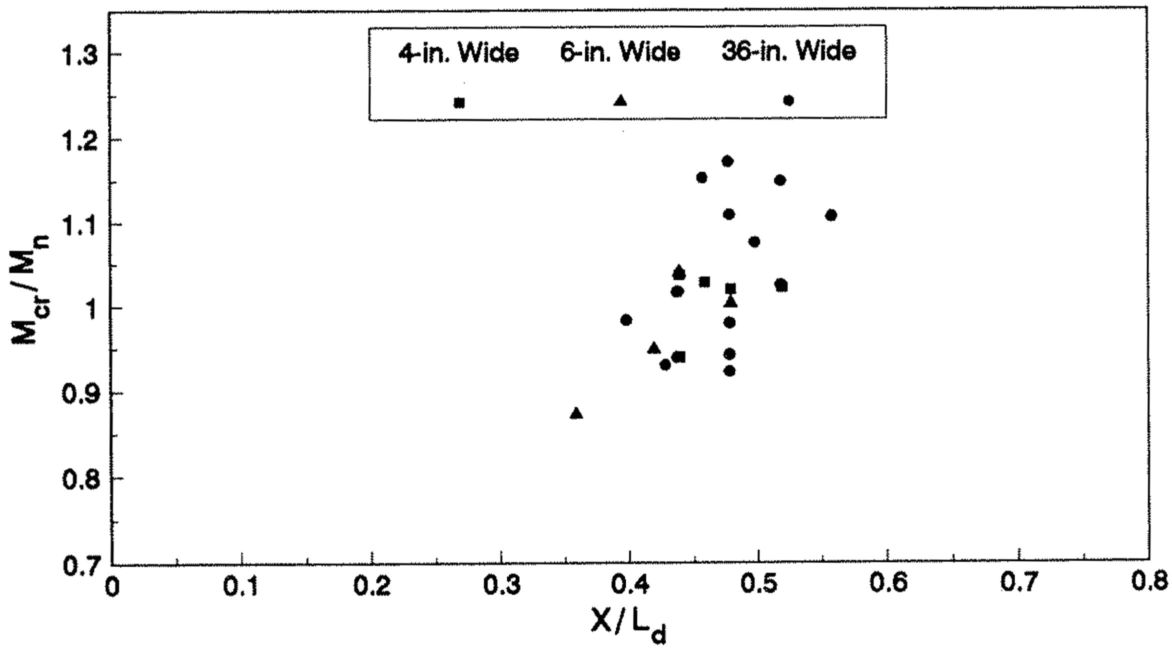
Figure 5.13. (continued)

occurs below this horizontal line, that particular specimen should have failed in either a bond or shear failure mode; and if a data point occurs above the horizontal line, a flexural failure mode of that particular specimen should have occurred. For a particular D-type specimen, a comparison of the actual failure mode and the predicted failure mode based on the location of the nondimensionalized data point with respect to the horizontal line in Fig. 5.12 or 5.13 normally revealed good agreement. For the specimens that had an actual failure mode that did not match the predicted failure mode, the difference was attributed to the anticipated scatter associated with experimental tests.

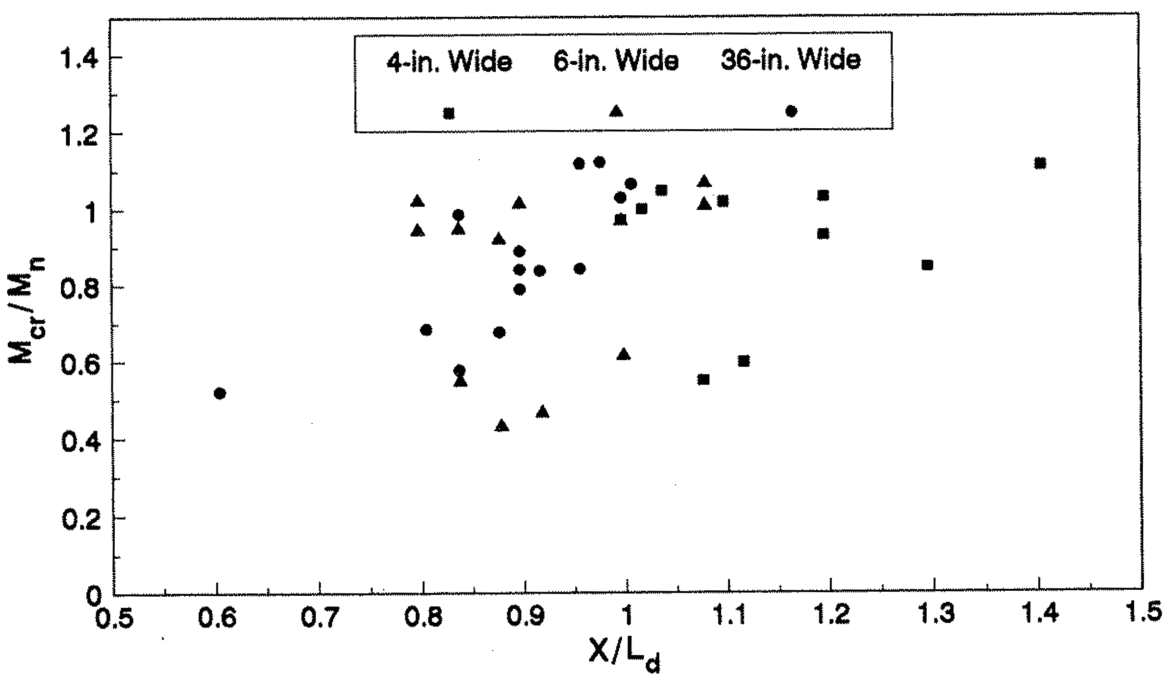
The strand development length can be qualitatively evaluated by either a visual inspection of the nondimensionalized results or by applying a straight-line extrapolation or interpolation through two or more of the nondimensionalized data points for the specimens that essentially had the same cross-sectional dimensions and strand surface coating. When many test results are available, the experimentally derived strand development length should be reasonably accurate. Ideally, an extrapolation or interpolation should only involve the data points that have the larger abscissa values and that correspond to specimens that failed by strand-slip (bond failure mode) when the ratio of M_{cr} to M_n was close to unity. However, when a limited number of data points are available for a particular configuration of the test specimens, the results obtained from similar specimens that experience a flexural failure may need to be incorporated. A straight line interpolation of the nondimensionalized data points shown in Fig. 5.12(a) and (c) revealed that the ratio X/L_d was equal to approximately 0.5 for the 4 and 36-in. wide specimens that were prestressed with coated strands. A visual inspection of Fig. 5.12(b) shows that one of the development length tests on the 6-in. wide, coated, single-strand specimens terminated with a flexural failure and the other three test specimens experienced a shear failure. However, the results shown in this figure appear to confirm the 0.5 value for the ratio X/L_d. For the three sizes of specimen cross sections that were tested, Fig. 5.12 shows that the ratio X/L_d was considerably less than unity. Therefore, the development length for the coated strands tested was substantially shorter than the length predicted by applying the AASHTO Specification [1] Eq. 9-32 [Eq. (4.28)]. Also, the three sizes of Type-D specimen cross sections did not significantly affect the coated-strand development length.

The uncoated-strand development length was established from the nondimensionalized data points for the specimens that failed in bond. Figure 5.13 shows a best-fit straight line that has been drawn through several of the nondimensionalized data points. An upper-bound value for the ratio X/L_d occurred at the intersection point of the best-fit straight line and a horizontal line drawn through the ordinate value for M_{cr}/M_n that was equal to unity. This graphical construction for the 4, 6, and 36-in. wide specimens produced X/L_d-values of 1.43, 1.08, and 1.03, respectively. Therefore, the development lengths for the uncoated strands tested was about 43% 8%, and 3% longer than the lengths predicted by applying the AASHTO Specification [1] Eq. 9-32 [Eq. (4.28)]. When the concrete side cover was small, the AASHTO Specification expression substantially underestimated the strand development length for the uncoated strands used in this research; and when adequate side cover is present, reasonably accurate uncoated-strand development lengths were predicted by the AASHTO Specification expression. Similar conclusions for narrow specimens have been made by Lane [31], Deatherage et al. [17], Cousins et al. [13], and Shahawy et al. [43].

Figure 5.14 shows the nondimensionalized data points for all of the coated and uncoated strand specimens. A comparison of the test results for the 4 and 6-in. wide D-type specimens clearly indicates that the amount of concrete side cover for the single-strand specimens tested did not affect the coated-strand development length but substantially affected the uncoated-strand development



(a)



(b)

Figure 5.14. Nondimensionalized relationship of moment strength versus load position for 4, 6, and 36-in. wide specimens: (a) coated strands; (b) uncoated strands

length. The difference in behavior between coated and uncoated strands has been attributed to the bond mechanism between the strand surface and the surrounding concrete, as discussed in Section 4.1. A comparison of the test results for the 6 and 36-in. wide specimens indicates that the spacing of the strands and the WWF used in the D-type specimens did not significantly affect the development length for either the coated or uncoated strands. Therefore, with regard to experimentally establishing the strand development length, the 6-in. wide, single-strand, D-type specimens reasonably represented the 36-in. wide, multiple-strand, D-type specimens.

The nondimensional parameters for shear strength (V_u/V_n) and load position (X/L) for the specimens listed in Tables 5.20 through 5.25 are plotted in Fig. 5.15(a) and (b) to show the relationship between shear strength and load position for the coated and uncoated-strand specimens, respectively. A horizontal line has been drawn through the V_u/V_n-ordinate value that equals unity. This line corresponds to the strength condition for which the shear force that was induced by the transverse load was equal to the nominal shear strength of the member. Theoretically, whenever a data point occurs below this horizontal line, the failure for that particular specimen should not be based on the shear strength of the member; and if the data point occurs above this line, a shear failure should have occurred in the member. The experimental test results of the coated-strand, D-type specimens revealed that 19 out of 24 tests actually involved a shear failure. These test results are identified by the solid symbols shown in Fig. 5.15(a). The tests that involved a bond and/or flexural failure and not a shear failure are identified by the hollow symbols. Figure 5.15(a) shows that all of the coated-strand, D-type specimens had a ratio of V_u to V_n almost equal to or greater than unity, and Fig. 5.15(b) shows that all of the uncoated-strand, D-type specimens had a ratio of V_u to V_n that was considerably smaller than unity. None of the specimens that contained uncoated strands failed in shear.

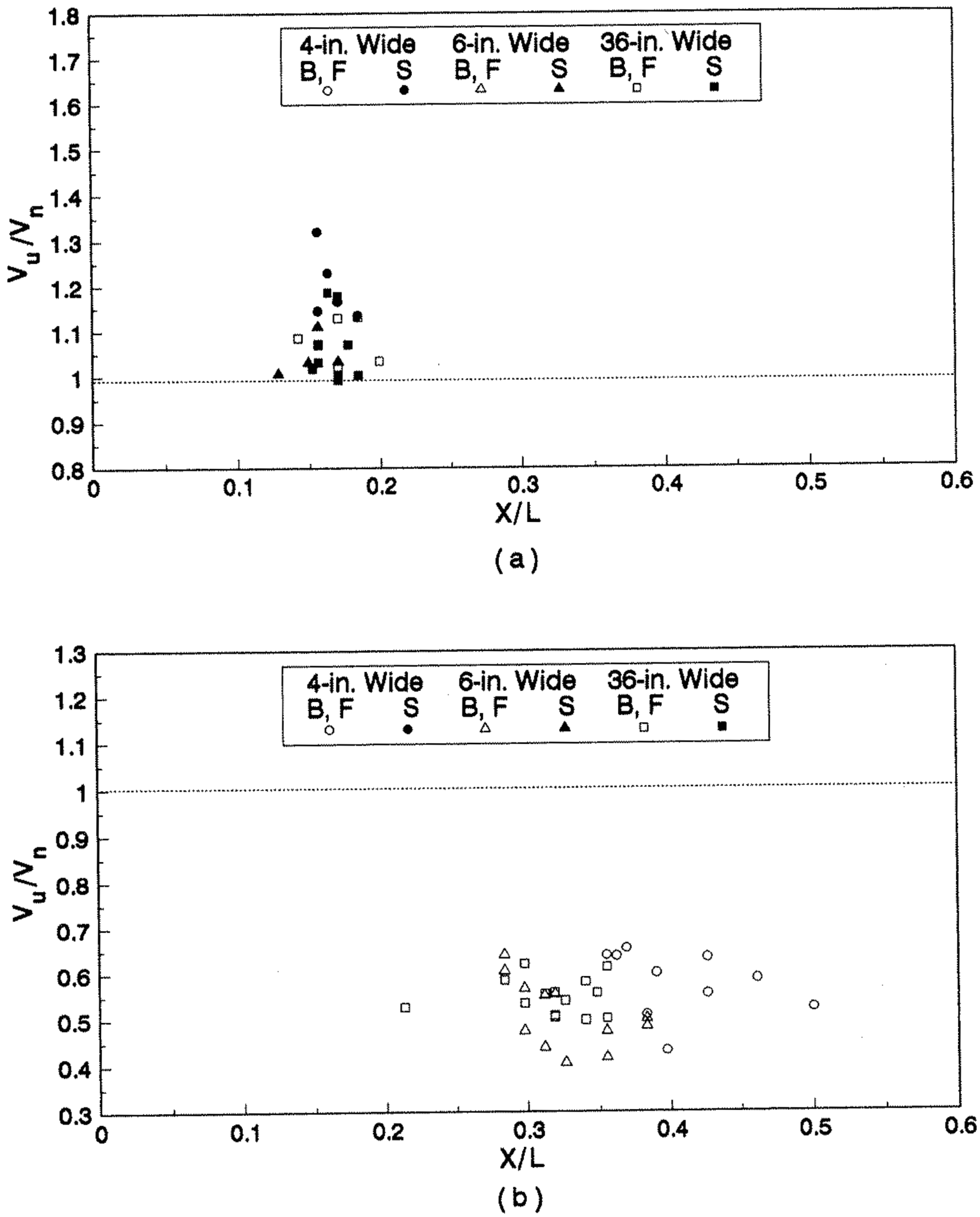


Figure 5.15. Nondimensionalized relationship of shear strength versus load position for 4, 6, and 36-in. wide specimens: (a) coated strands; (b) uncoated strands

5.5.4. Comparisons with Other Researchers

The development length of coated and uncoated prestressing strands have been experimentally investigated by other researchers. Table 5.26 lists the 3/8-in. diameter, seven-wire, 270-ksi, prestressing strand development lengths that were measured in the research reported herein, by Cousins et al. [13], and by Mitchell et al. [37]. The strand type notation C-LR, U-LR, and U-SR listed in the table are abbreviations for coated low-relaxation, uncoated low-relaxation, and uncoated stress-relieved prestressing strands, respectively. Since small variations in the concrete strength f'_{cd} obtained in this research did not appear to significantly affect the strand development lengths, the average strengths f'_{cd} and f^*_{su} and average value of L_d for the specimens that were essentially identical have been listed in Table 5.26. A comparison of the strand development lengths for the 4-in. wide by 6-in. thick specimens revealed that the lengths obtained from this research were comparable to the lengths obtained by Cousins et al. [13] for both the coated and uncoated strands.

To determine whether anlaytical models predict the development lengths for the coated and uncoated strands used in this research, the researchers compared the measured and calculated strand development lengths. The theoretical expressions that were applied for the strand development length have been presented by Cousins et al. [14] for Eq. (4.30), the PCI Ad Hoc Committee on Epoxy-Coated Strands (PCI Guidelines) [40] for Eq. (4.27), the AASHTO Specifications [1] and ACI Building Code [3] for Eq. (4.27), Zia and Mostafa [45] for Eq. (4.28), and Mitchell et al. [37] for Eq. (4.29). In the PCI Guidelines, the development length expression for coated strands is identical to the ACI Building Code uncoated-strand development length expression. Table 5.27 lists the measured strand development lengths for the 4, 6, and 36-in. wide, D-type specimens that contained

Table 5.26. Comparisons of measured development lengths for 3/8-in. diameter, seven-wire, 270-ksi, prestressing strands

Researcher	Strand Type	Specimen Size (in.)		Average f'_{cd} (psi)	Average f^*_{su} (ksi)	Average L_d (in.)
		b	h			
Abendroth, Stuart, Yuan	C-LR	4 6 36	6 6 6	5140 5140 5600	264 266 266	24 24 26
Cousins et al. [12,14]	C-LR	4	6	5340	253	24
Abendroth, Stuart, Yuan	U-LR	4 6 36	6 6 6	5690 5830 4800	264 266 266	56 50 49
Cousins et al. [12,14]	U-LR	4	6	5340	253	57
Mitchell et al. [37]	U-SR	3.9	7.9	4500	263	47[a]

[a]Strand surface was slightly rusted

Table 5.27. Comparisons of measured and calculated development lengths for 3/8-in. diameter, seven-wire, 270-ksi, low-relaxation, grit-impregnated, epoxy-coated strands

Cast No.	Specimen Size (in.)		f'_{ci} (psi)	f'_{cd} (psi)	f_{si} (ksi)	Average Stress		Strand Development Length (in.)			
	b	h				f_{se} (ksi)	f^*_{su} (ksi)	Abendroth, Stuart, Yuan (measured)	Cousins et al [14] (Eq. 4.30)[b]	AASHTO [1] & ACI [3] (Eq. 4.27)	PCI Guidelines [40] (Eq. 4.27)
17	4	6.0	4390	5130	202.5	196.6	264.7	22-24	25.5	50.1	50.1
17	6	6.0	4390	5130	202.5	198.8	266.0	< 24	25.5	50.1	50.1
8	36	6.0	4150	5120	202.5	198.7	266.3	26	25.9	50.2	50.2
9	36	6.0	4670	5710	202.5	198.9	266.5	25	24.7	50.2	50.2
12	36	6.0	4420	6140	202.5	198.8	266.7	24	24.7	50.3	50.3
14	36	6.0	4240	5230	202.5	198.7	266.3	> 24	25.7	50.2	50.3
Average	-	-	4377	5410	202.5	198.4	266.1	25[a]	25.3	50.2	50.2
Nominal	4	6.0	4000	5000	202.5	197.1	242.9	-	22.7	41.8	41.8
	6 or 36	6.0	4000	5000	202.5	198.9	251.9	-	23.9	44.7	44.7

[a]Used upper limits for Cast No. 17 and lower limit for Cast No. 14
[b]$U'_t = 16.5$ for medium to high grit density

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coated strands; concrete stresses f'_{ci}, and f'_{cd}; strand stresses f_{si}, which were assumed to be equal to 0.75 f'_s; and average values of f_{se} and f^*_{su} for the specimens in a particular concrete casting. The calculated, coated-strand development lengths listed in Table 5.27 were evaluated by applying the experimentally based stresses in the expressions given by Cousins et al. [14], AASHTO Specifications [1] and ACI Building Code [3], and PCI Guidelines [40].

The predicted, coated-strand development lengths established by the expressions given in the AASHTO Specifications, ACI Building Code, and PCI Guidelines were about twice as long as the measured lengths. The reason for the large difference between the computed and measured lengths is that the emperical expression [Eq. (4.27)] for the strand development length that was adopted by these three associations was based on test results for uncoated prestressing strands. The measured, coated-strand development lengths were closely predicted by applying the expression presented by Cousins et al. [14] for Eq. (4.30). Table 5.27 also lists nominal, coated-strand development lengths that were calculated by using nominal stresses. The nominal stress f_{si} was equal to 0.75 f'_s and the nominal stress f_{se} and f^*_{su} were computed from Eq. (4.12) and (4.19), respectively. The nominal, coated-strand development lengths were always less than the lengths established by applying the experimental stress, since the stresses f^*_{su} were always higher and the stresses f_{se} were about the same as their nominal values.

Table 5.28 lists the measured and calculated development lengths for the 4, 6, and 36-in. wide, D-type specimens that contained uncoated strands. The table also lists the stresses needed to evaluate the expressions presented by Cousins et al. [14], the AASHTO Specifications [1] and ACI Building Code [3], Zia and Mostafa [45], and Mitchell et al. [37]. The predicted, uncoated-strand development lengths for the specimens within each concrete casting were significantly overestimated

Table 5.28. Comparisons of measured and calculated development lengths for 3/8-in. diameter, seven-wire, 270-ksi, low-relaxation, uncoated strands

Cast No.	Specimen Size (in.)		f'_{ci} (psi)	f'_{cd} (psi)	f_{si} (ksi)	Average Stress		Strand Development Length (in.)				
	b	h				f_{se} (ksi)	f^*_{su} (ksi)	Abendroth, Stuart, Yuan (measured)	Cousins et al [14] (Eq. 4.30)	AASHTO [1] & ACI [3] (Eq. 4.27)	Zia & Mostafa [45] (Eq. 4.28)	Mitchell et al [37] (Eq. 4.29)
10	4	6.0	4050	5350	202.5	196.4	264.8	65-70	85.1	50.2	55.6	44.4
11	4	6.0	4730	6150	202.5	196.8	265.1	51	79.2	50.2	51.5	41.3
10	6	6.0	4050	5350	202.5	198.6	266.1	54	84.8	50.1	55.2	44.4
11	6	6.0	4730	6150	202.5	198.9	266.3	45	78.9	50.2	51.1	41.2
6	36	6.0	2910	2920	202.5	197.9	264.3	40-50	107.3	49.7	65.7	55.8
7	36	6.0	3980	4890	202.5	198.6	266.2	45	87.4	50.2	55.7	45.7
15	36	6.0	4010	5440	202.5	198.6	266.4	48	84.7	50.2	55.6	44.4
16	36	6.0	4780	5870	202.5	198.9	266.6	49	80.0	50.2	50.9	41.7
Average			4333[b]	5600[b]	202.5	198.1	265.7	50[a,b]	82.9[b]	50.1	53.7[b]	43.3[b]
Nominal	4	6.0	4000	5000	202.5	197.3	242.9	-	69.5	41.8	45.3	37.4
	6 or 36	6.0	4000	5000	202.5	199.0	251.9	-	75.5	44.7	48.7	40.1

[a]Used upper limit on Cast No. 10
[b]Cast No. 6 was omitted due to low concrete strength

for all cases by the expression in Cousins et al. [Eq. (4.30)], underestimated for the 4-in. wide specimens and generally accurate for the 6 and 36-in. wide specimens by the expression in the AASHTO Specifications and ACI Building Code [Eq. (4.27)], reasonably accurate for the 4 and 6-in. wide specimens and slightly overestimated for 36-in. wide specimens by the expression in Zia and Mostafa [Eq. (4.28)], and slightly to significantly underestimated for all cases by the expression in Mitchell et al. [Eq. (4.29)]. Comparisons were made between the nominal uncoated-strand development lengths, which were computed using the nominal stresses, and the measured uncoated-strand development lengths. The nominal, uncoated-strand development length that was calculated by applying the expression by Cousins et al. [14] moderately to substantially overestimated the measured lengths. The nominal length established by applying the expression in the AASHTO Specification [1] and the ACI Building Code [3] underestimated the measured lengths. The nominal length established by applying the expression by Zia and Mostafa [45] significantly underestimated the measured lengths for the 4-in. wide specimens and reasonably predicted the measured lengths for the 6 and 36-in. wide specimens. The nominal length established by applying the expression by Mitchell et al. [37] significantly underestimated the measured lengths in all cases.

5.6. Stand Seating at End Chuck

Figure 5.16 shows a typical strand prestress force versus strand displacement relationship for a coated strand (Strand No. 5 in Cast No. 14) and an uncoated strand (Strand No. 3 in Cast No. 10) that were measured at an anchorage-end prestressing chuck during strand tensioning. To eliminate the effects of initial seating of a strand in the jaws of a chuck, the researchers applied a prestress force

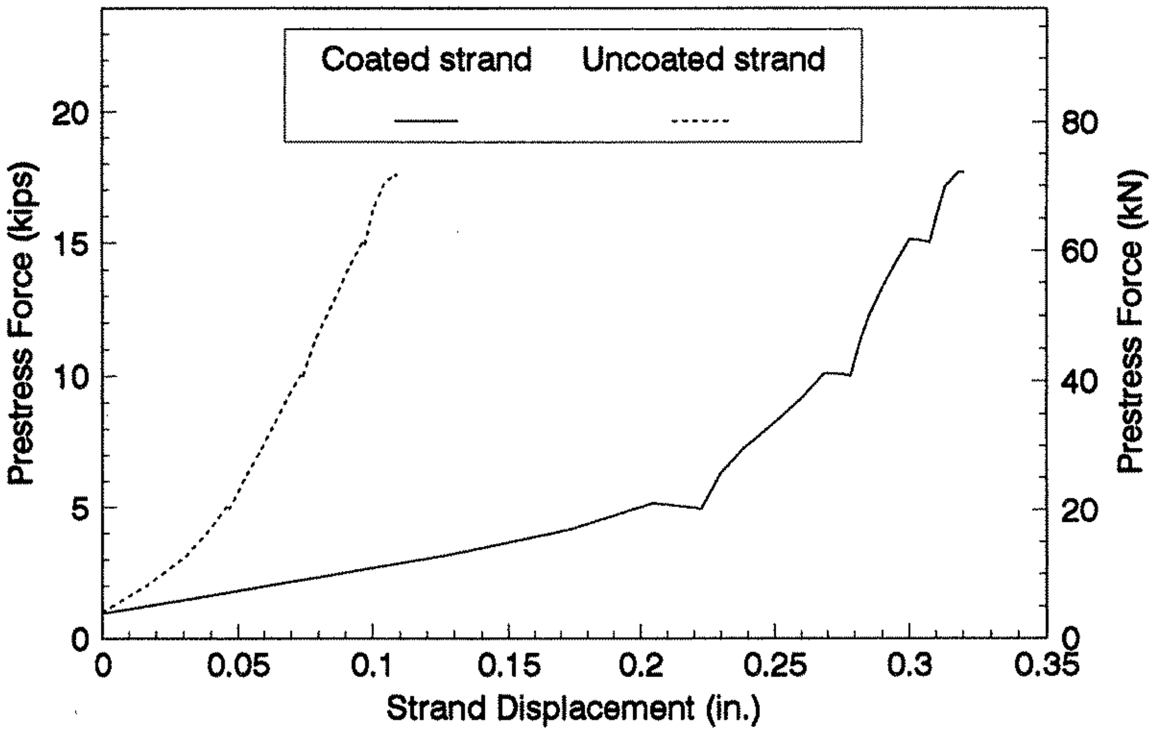

Figure 5.16. Movements at anchorage chucks during strand tensioning for coated Strand No. 5 in Cast No. 14 and uncoated Strand No. 3 in Cast No. 10

of 1,000 lbs before measurements of strand movement were recorded. As shown in the figure, when the prestress force in the strand increased, the strand movement into the chuck also increased; however, the relationship between force and displacement was not linear. As the teeth on the jaws of the chuck engaged the outer wires of the strand, the rate of axial displacement decreased. Strand movements into some of the chucks at the prestressing end were also measured. These movements were nominal displacements since the coupling assembly between a strand and a prestressing bar did not permit an attachment of a DCDT at the free end of a strand. The strand movement (not shown) at this end of the strand was similar to the displacement at the other end of the strand. When the prestress in a strand was equal to 75 percent of the ultimate strand tensile strength, which corresponded to a prestress force of 17.2 kips in a 3/8-in. diameter, 270-ksi, low-relaxation strand, the movement of a strand at the anchorage-end chuck was approximately equal to 0.31 and 0.11 in. for the coated and uncoated strands, respectively. As Fig. 5.16 shows, the displacements for a coated strand were substantially larger than those for an uncoated strand, especially during the initial portion of the strand prestressing when the forces were low. For a coated strand, the length of the chuck and the depth and size of the teeth in the jaws of the chuck are larger than those for an uncoated strand, since the teeth need to penetrate the epoxy coating to grip the outer steel wires of the strand. Figure 5.17(a) and (b) shows several strand force versus strand displacement relationships for coated and uncoated strands, respectively. The load versus displacement behavior was quite consistent for both the coated and uncoated strands.

Table 5.29 lists the maximum strand displacements at the anchorage-end chucks for the monitored coated and uncoated strands when a prestress force that corresponded to 75% of the

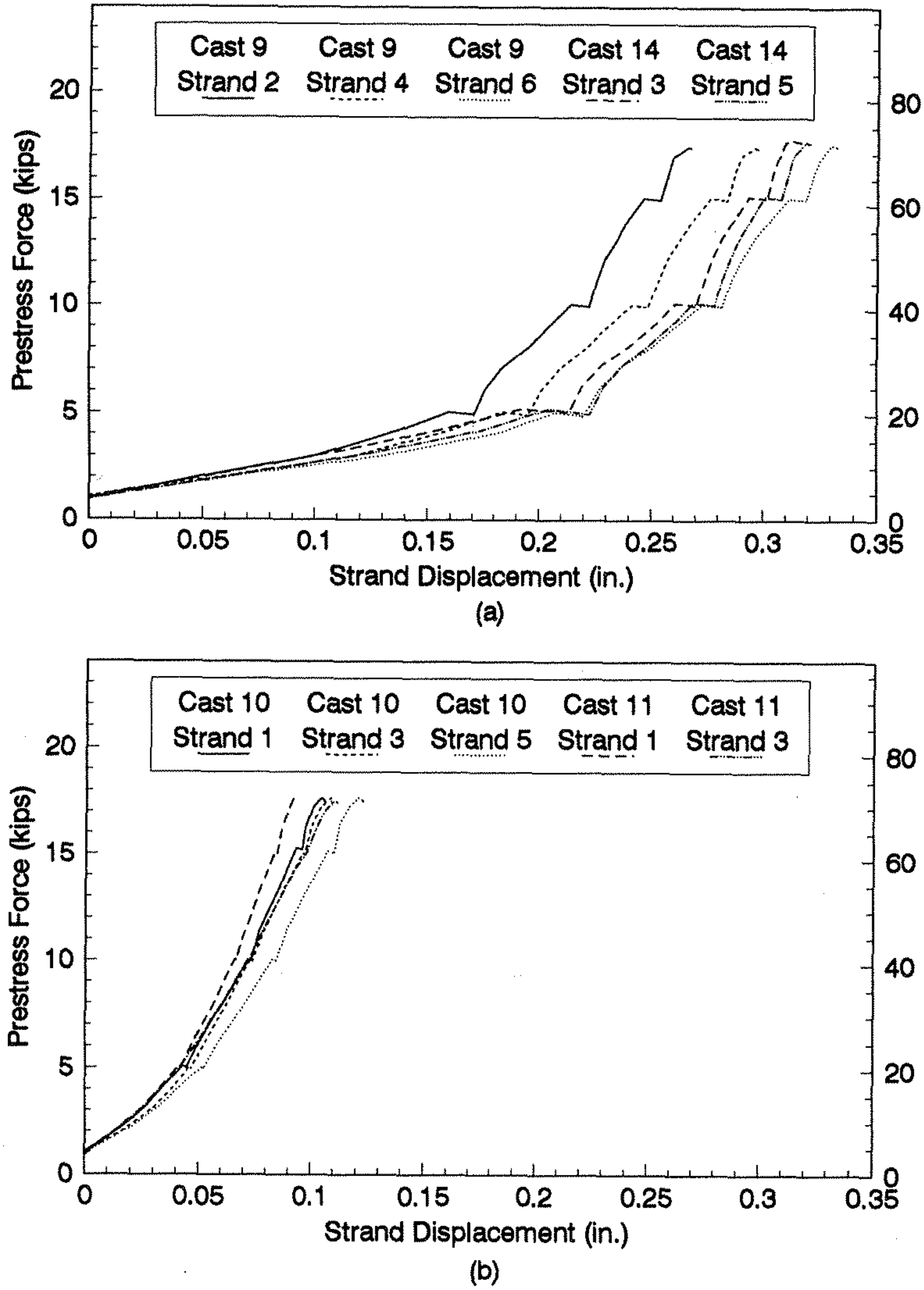

Figure 5.17. Movements at the anchorage chucks during strand tensioning for: (a) Coated strands; (b) Uncoated strands

Table 5.29. Strand displacements at anchor-end chucks when the prestress force was equal to 17.2 kips

Coated Strands			Uncoated Strands		
Cast No.	Strand No.	Displacement (in.)	Cast No.	Strand No.	Displacement (in.)
9	2	0.274	10	1	0.106
	4	0.302		3	0.109
	6	0.341		5	0.122
14	3	0.323	11	1	0.093
	5	0.325		3	0.113
Average		0.313	Average		0.109

ultimate tensile strength of the strand was applied. The average displacement for the five coated and uncoated strands was equal to 0.313 and 0.109 in., respectively. The coated strands experienced almost three times the amount of movement that occurred with the uncoated strands. This displacement difference did not present any difficulties during the experimental portion of the research reported here due to the method used to prestress the strands. When strands are tensioned in a more conventional fashion (without a coupler assembly and a post-tensioning bar), the larger strand seating displacements for the coated strands compared to those for uncoated strands would have to be considered.

After the strands were prestressed, the DCDTs continued to monitor any relative movement between the strands and the chucks. Slippage of the strands into the chucks was not detected for either the coated or uncoated strands. Therefore, once the required strand prestress was reached, the chucks held the force until the strands were detensioned.

When a strand was detensioned with an abrasive grinding wheel, the epoxy coating broke-off from the exposed end portion of the strand as the outerwires of the strand unwound from their original positions. The unwound strand length was not measured; however, this length did not extend to the end of the PC specimen which was about 10 to 12 in. from the strand cutting location. If the header depth in a prestressing bed is smaller than the 24-in. depth used in this research, the epoxy coating might break-off to the face of a specimen when the strand is detensioned. If strand extensions beyond the ends of a PC panel are required, the outer strand wires will need to be rewrapped around the center strand wire, and an epoxy coating will have to be reapplied to any portion of an exposed bare strand in order to maintain the corrosion rsistance of the strand.

To verify that the coated strands had been gripped properly by the chucks, the researchers placed the end portions of some of these strands into an oven to burn-off the epoxy coating. After the epoxy coating had been removed, an inspection of these strand segments revealed that notches had been cut into the outer steel wires of the strand. Therefore, the teeth in the jaws of the chucks had penetrated the epoxy coating to grip the outer strand wires. As expected, an inspection of the uncoated strands also revealed that the outer wires of these strands had gripped by the chucks.

6. EPILOGUE

6.1. Summary

6.1.1. Overview

Composite bridge decks, which contain thin precast prestressed concrete (PC) subdeck panels and a cast-in-place reinforced concrete (RC) topping slab, have been used as an alternate to monolithic, full-depth, RC bridge decks. Currently (1994), the steel reinforcement in the PC subdeck panels consists of uncoated prestressing strands that are located at the midthickness of the panels and uncoated welded wire fabric (WWF) that is positioned on the top of the strands. To improve the corrosion resistance of the panel reinforcement, the Iowa Department of Transportation has proposed the substitution of epoxy-coat reinforcement for the uncoated reinforcement. This study reported herein was conducted to investigate the feasibility of using grit-impregnated, epoxy-coated, prestressing strands and smooth-surfaced, epoxy-coated, WWF in PC subdeck panels.

The primary objective for Phase I of the research on epoxy-coated reinforcement for PC panels was to establish a recommended minimum thickness for PC bridge subdeck panels reinforced with 3/8-in. diameter, seven-wire, 270-ksi, low-relaxation, grit-impregnated, epoxy-coated, prestressing strands (coated strands) and smooth-surfaced, epoxy-coated WWF to prevent through-thickness concrete cracking when the prestress force is transferred to the concrete. Other objectives of the study included the evaluation of the short-term bond performance of coated and uncoated strands and the seating characteristics of the wedge-shaped grips that are used to anchor coated and uncoated strands.

To accomplish these objectives, the researchers performed a literature review of the research reported on coated and uncoated prestressing strands; conducted a survey of design agencies and PC member producers; completed an extensive test program that involved the construction of 115 test specimens for conducting panel thickness investigations, strand transfer length tests, and strand development length tests; and performed analytical studies of strand transfer and development lengths.

6.1.2. Literature Review

The study on bond characteristics of prestressing strands started in the late 1950s. Since then, most of the research has involved uncoated prestressing strands and only a few studies have addressed epoxy-coated strands. As of December 1994, studies that have investigated the transfer and development lengths and performance of coated strands in thin PC bridge subdeck panels have not been found in the literature. The main subjects of the previous research on prestressing strands addressed transfer length and development length studies of different types of strands, parameter influences on strand transfer and development lengths, and analytical models of strand transfer and development lengths.

Several conclusions associated with strand transfer and development lengths were formulated from the previous research of coated and uncoated strands.

- Transfer and development lengths of prestressing strands increase as the nominal strand diameter increases.
- Difference in the transfer lengths for 250 and 270-ksi strands was not significant.
- Strands with a rough surface have shorter transfer lengths than those with a smooth surface.
- Concrete type (normal-weight or lightweight) has a negligible effect on strand transfer length.

- Cyclic loading has a negligible effect on strand transfer and development lengths.
- Epoxy-coated strands have good corrosion resistance.
- Smooth surface epoxy-coated strands cannot develop sufficient bond strength, while grit-impregnated, epoxy-coated strands can develop adequate bond strength.
- Grit-impregnated, epoxy-coated strands have shorter transfer and development lengths than comparable uncoated strands.
- Transfer and development lengths of grit-impregnated, epoxy-coated strands decrease as the grit density increases.
- Sudden releases of the strand prestress forces results in longer transfer lengths than that caused by gradual releases.
- Strand development length was greatly influenced by shear and confinement reinforcement.
- Higher concrete compressive strengths produced shorter transfer and development lengths for uncoated strands; however, within a small range of strength variation the difference in strand transfer and development lengths was not sufficient.
- Elevated temperatures greatly affect the bond strength of epoxy-coated strands. Bond strength reductions begin at a temperature of about 125° F and bond strength is essentially completely lost at a temperature of about 200° F.
- Concrete splitting failures were observed when the prestress force was transferred to the concrete in some small cross-section specimens that contained a single, grit-impregnated, epoxy-coated strand.

6.1.3. Questionnaires

A survey questionnaire was distributed to bridge engineers in the 50 state departments of transportation; 3 branches of U.S. Forest Service; 9 Canadian provinces, Northwest Territories, and Puerto Rico transportation agencies; New Jersey Turnpike, New York State Bridge and New York Thruway Authorities; and Port Authority of New York and New Jersey. The questions in the survey were related to the general background of the design agency, types of epoxy-coated reinforcement, epoxy coatings, range of application for coated reinforcement, design with epoxy-coated prestressing strands, experience with epoxy-coated strand, and epoxy-coated reinforcement details and specifications.

Fifty-three (90%) of the design agencies which returned the survey stated that they allow or have allowed the use of epoxy-coated reinforcement in bridge structures. Fifty-two of these design agencies have used epoxy-coated standard deformed bars, while only 3, 4 and 16 of the 53 agencies have used epoxy-coated prestressing bars, prestressing strands, and WWF, respectively. All four agencies that have had experience in using epoxy-coated strands require grit-impregnated, epoxy-coated strands and only one of these four design agencies has used coated strands in PC subdeck panels.

The most common type of coated strands used are seven-wire, 270-ksi, low-relaxation, prestressing strands. Among the four design agencies that have used epoxy-coated prestressing strands, three agencies have specified 1/2-in. diameter strands and one agency has specified 3/8-in. diameter strands. The minimum amount of concrete cover over the coated strands in PC panels or slabs is between 1 and 1-3/4 in. The minimum center-to-center spacing between individual coated strands is either 2 in. or 6 in. Two design agencies specify that confinement reinforcement be used along the strand development length in PC panels or slabs. Three agencies apply the AASHTO Specification [1] expression for uncoated strands to calculate the development length for epoxy-coated prestressing strands, and the other agency was uncertain as to how to evaluate the development length for coated strands.

When the representatives from these four design agencies were asked to classify any problems associated with their usage of epoxy-coated strands and to rate the usage of coated strands, all four respondents replied that they could not answer these questions because of the limited experience that they have had with coated strands. Some of the additional comments received from the agencies that returned the questionnaire were as follows: usually sufficient concrete cover should eliminate the need for coated strands even if the bridge decks are exposed to salt; the extra cost of epoxy-coated strand will probably prevent significant use of these strands; coated strands have performed successfully since 1985 in a bridge structure; and our agency supports the use of epoxy-coated strands in bridge girders.

A questionnaire similar to the one sent to the design agencies was also distributed to 205 PC producers who are members of the Prestressed/Precast Concrete Institute. Seventy-six (about 37%) of these questionnaires were returned. Of those manufacturers that returned the survey, 57 have produced PC members with epoxy-coated reinforcement, and 41 have used epoxy-coated reinforcement in bridge structure members.

Out of the 57 precastors who have used epoxy-coated reinforcement in their products, 54 companies have used epoxy-coated standard deformed bars, 2 have used coated prestressing bars, 13 have used coated prestressing strands, 20 have used coated WWF, and 2 companies have used coated spiral wire. Three PC producers have used epoxy-coated strands in bridge girders, hollow core slabs, and piles; two companies have used them in full-depth bridge deck panels; and only one company has used them in single or double-tee sections. The precastors that have used coated prestressing strands noted that Florida Wire & Cable Company is their only epoxy-coated strand supplier.

Among the 13 companies that have used coated strands, the 1/2-in. diameter, seven-wire, 270-ksi, low-relaxation strand is the size most commonly used. Only one precastor has used 3/8-in. diameter coated strands. When coated strands were used in PC slabs or panels, the minimum concrete cover over a strand was specified to be between 1 to 2 in., and the minimum strand spacing ranged between 2 to 8 in. Four of the five producers that make PC slabs or panels place confinement reinforcement along the development length of the coated strands.

Each manufacturer was asked to state any specific problems that they have experienced with epoxy-coated strands and concrete members reinforced with these strands. Some manufacturers were not able to provide comments because of their limited usage of coated strands. Seven of the 13 producers that have used coated strands listed their problems as: slippage of strands at the end chucks, difficulties in removing the chucks from the strand ends after cutting, and increased difficulty of handling coated strands over uncoated strands. However, no producers categorized these problems as major problems. When the producers were requested to rate the usage of epoxy-coated strands considering all aspects of manufacturing and performance of members, five producers rated them as fair, one chose good, and another one chose very good. Some of the additional comments made by the producers included: *chuck seating requires more strand movement for epoxy-coated strands; steam curing could be a problem with coated strand since the coating softens at about 150°F; and caution should be taken for using coated strand when fire resistance is desired.*

6.1.4. Experimental Tests

A total of 115 PC test specimens were constructed during 17 concrete castings in a 54-ft long by 9.3-ft wide prestress bed that was fabricated from salvaged steel bridge girder members. The location of the transverse steel headers in this frame, the elevation of the bottom surface of the prestress bed, and the location of the wood sideforms were adjusted to match the desired specimen dimensions and to position the prestressing strands. The specimens were prestressed with 3/8-in. diameter, seven-wire, low-relaxation strands. Sixty-seven of the specimens contained grit-impregnated, epoxy-coated strands (coated strands) and the remaining 48 specimens contained bare strands (uncoated strands). Two types of specimens were cast. Seventy-five transfer length (T-type) specimens were tested to establish the recommended minimum thickness for thin PC panels that were prestressed with coated strands and to measure the transfer lengths of the coated and uncoated strands. Forty development length (D-type) specimens were tested to measure the development lengths of the coated and uncoated strands.

Both single-strand and multiple strand specimens were cast. For the T-type specimens, the strands were located at the midthickness of the specimens; and for the D-type specimens, the strands were positioned at two-thirds of the depth from the top surface. The multiple-strand specimens contained either two or six strands spaced at 6 in. on center. Eight sizes of T-type specimens were cast, and three sizes of D-type specimens were cast. Some of the 36-in. wide, T-type specimens contained a layer of 6 x 6 - D6 x 6 WWF that was placed, with the longitudinal wires of the fabric above the transverse wires, directly on top of the six prestressing strands. Also, these specimens had a raked top concrete surface to simulate a PC subdeck panel.

Except for Cast No. 13, the 3/8-in. diameter strands were prestressed to about 75% of their guaranteed ultimate tensile strength. For the 270-ksi strands, this level of prestress corresponded to a force of about 17.2 kips per strand and a stress of about 202.5 ksi. The coated strands in Cast No. 13 were intentionally overstressed to produce a force of about 18.9 kips per strand, which corresponded to a strand stress of about 222 ksi or about 82% of the ultimate tensile strength of a strand. The larger prestress force was applied to the specimens in this casting in order to confirm that the panel thickness was adequate to prevent concrete splitting failures during the specimen prestressing procedures. The tension forces in the prestressing strands were obtained from strains that were measured by strain gages. These gages were attached to the post-tensioned bars which were used to pull the strands. Also, the strand forces were measured by load cells during the strand tensioning procedure.

The concrete mix design used for the specimens was modeled after the mix design that is used to construct PC subdeck panels at Iowa Prestress Concrete in Iowa Falls, Ia. The mix design satisfied the Iowa DOT Specification [21] requirements. The minimum concrete compressive strength just prior to cutting the prestressing strands was 4000 psi, and the minimum 28-day compressive strength was 5000 psi. The mix design actually produced the 4000 psi compressive strength when the concrete was about one to two days old and the 5000 psi strength was reached about two to three days later. The testing of the T-type and D-type specimens was conducted when the concrete compressive strength was 4000 and 5000 psi, respectively. All of the concrete for the specimens was ordered from two local ready-mix concrete suppliers.

After the concrete was cast, the specimens were moisture cured for minimum of 24 hours and then left to air dry until the strands were released. After the concrete compressive strength had reached 4000 psi, the prestressing strands were detensioned with an abrasive grinding wheel at the locations of the steel headers in the prestress frame. A strand cutting sequence was developed to minimize the eccentric compressive loading on the specimens.

The recommended minimum thickness of PC subdeck panels that are prestressed with coated strands was experimentally established by casting and prestressing T-type specimens of different thicknesses. If the thickness was too thin, concrete cracks developed in the top and/or bottom surfaces of the specimens during and after cutting the strands. These cracks were directly above and/or below one or more of the coated strands. The smallest specimen thickness that was required to prevent the formation of any visually detectable concrete cracks in any of the specimens with that thickness was selected as the minimum recommended thickness for PC subdeck panels containing coated strands and epoxy-coated WWF.

The strand transfer length is the length of strand embedment in the surrounding concrete that is required to develop the effective strand prestress. To measure the transfer length, electrical resistance strain gages were embedded into some of the T-type specimens between adjacent strands or between the outside strand and wood sideform at the midthickness of the specimens. Induced axial concrete strains due to prestressing the specimens were recorded as the difference in the strain reading just prior to and just after strand cutting. The strand transfer length was established by analyzing the distribution of concrete strains along the specimen length.

The strand development length is the total strand embedment length in surrounding concrete that is required to develop the strand stress that occurs when the nominal moment strength of the PC member is reached. Cross bending tests of simply supported D-type specimens were conducted to experimentally establish the strand development lengths. These tests involved the application of a load across the width of a specimen at a selected distance from one end of the specimen. If the loading produced a flexural failure of a specimen, the next test on the opposite end of the same specimen or on a new specimen that was essentially identical to the previous specimen was performed with the load moved closer towards the near end of the specimen. However, if the loading produced a bond failure between a strand and concrete before a flexural failure, the load was moved further into the span for the next test. The strand development length was considered to be the distance from load position to closest end of a specimen for which the failure involved a transition between a flexural failure and bond failure. The load and the load-point deflection were recorded with a load cell and displacement transducers, respectively. Both ends of every strand were also monitored for slippage during the development length tests. A slip measurement of 0.01 in. was considered to correspond with the occurrence of a bond failure.

The strand seating displacement characteristics of the wedge-shaped grips for both coated and uncoated strands were evaluated by measuring the displacements between the end portion of the strands and the chucks with transducers. For four of the concrete castings, three strands were monitored during the strand tensioning process. To eliminate the initial effects of seating of a strand in the grips of a chuck, the researchers applied a prestress force of 1000 lbs before measuring strand movements.

6.1.5. Analytical and Experimental Results

Several material properties were determined by conducting experimental tests. The concrete compressive strengths, modulus of rupture strengths, and modulus of elasticity values were established from standard cylinder tests, standard beam prism tests, and strand transfer and development tests, respectively. Except for Cast Nos. 4 and 6, which contained concrete that was not representative of the concrete used in industrial construction of PC subdeck panels, the remaining 15 concrete castings contained concrete of acceptable quality. For these 15 castings, the concrete compressive strengths ranged from 3980 to 4780 psi when the strands were cut, from 4890 to 6150 psi when the development length tests were conducted, and from 5390 to 8430 psi when the concrte was 28-days old. For these same concrete castings, the modulus of rupture strengths ranged from 404 to 589 psi when the strands were cut and from 424 to 566 psi when the concrete was 28-days old. For Cast Nos. 10-12 and 14-17, the modulus of elasticity of the concrete ranged from 2760 to 4060 ksi when the strands were cut. For Cast Nos. 6-12 and 14-17, the modulus of elasticity of the concrete ranged from 2720 to 4850 ksi when the strand development length tests were conducted. Tension tests on the 3/8-in. diameter coated and uncoated strands revealed that the modulus of elasticity for the coated strand was 29,600 ksi and for the uncoated strand was 28,300 ksi, which were 2.3% higher and 0.8% lower, respectively, than the values provided by the strand manufacturer.

During some of the initial concrete castings, fluctuations in the strand forces were detected during the concrete curing period. To establish the reason for the changes in the strand forces, the researchers used thermocouples and resistance temperature devices to measure temperatures of the air, concrete, strand, and the prestress frame. This instrumentation revealed that before casting the concrete, the prestressing force in the tensioned strands changed due to moderate room temperature variations. After the concrete was cast, the strand temperature was affected by the temperature of the surrounding concrete. The heat of concrete hydration caused the strand temperature to rise for about eight hours, which resulted in a decrease of the prestressing forces in strands. Minimum strand forces occurred when the concrete temperature was a maximum. As the concrete temperature decreased, the prestressing forces in strands increased and approached the values close to those that occurred just prior to casting of the concrete. The maximum temperature recorded during all of the castings was about 114°F, and the maximum temperature when the strands were released was about 90°F. These temperatures were below a temperature of 125°F that has been suggested [34] as a threshold temperature for PC members containing coated strands.

During and after cutting the prestressing strands, the specimens were inspected for visible concrete cracks that may have formed as a result of prestressing the specimens. These inspections revealed that concrete cracks did not occur in any of the 2.5-in. thick specimens that were prestressed with uncoated strands. Concrete cracks were found in eight of the twelve 12-in. wide by 2.5-in. thick specimens that were prestressed with two coated strands, in all four of the 36-in. wide by 2.5-in. thick specimens that were prestressed with six coated strands, and in the one 36-in. wide by 2.5-in. thick specimen that contained six coated strands and coated WWF. Concrete cracks were not found in any of the twenty-four 12-in. wide by 3-in. thick specimens that were prestressed with two coated strands, and in any of the four 6-in. wide by 3-in. thick specimens that were prestressed with one coated strand, or in any of the six 36-in. wide by 3-in. thick specimens containing six coated strands and coated WWF. Even for Cast No. 13, for which the prestressing forces in the strands just prior to detensioning were about 10% higher than the normal prestressing forces for 3/8-in. diameter strands, visible concrete cracks were not detected in any of these 36-in. wide by 3-in. thick specimens containing six coated strands and coated WWF.

The strand transfer lengths were determined by applying a slope-intercept procedure to the graphs of the concrete axial strains that were measured with embedment strain gages in selected T-type specimens. The measured concrete strains increased essentially linearly from zero strain at the free end of a specimen to a relatively constant maximum strain that began at a certain distance from the end of a specimen. After constructing best-fit sloping and horizontal lines through the strain data points, the transfer length was obtained as the distance from the end of a specimen to the intersection point for the two best-fit lines. Strand transfer lengths were also calculated by applying emperical expressions that were obtained from the literature.

The strand development lengths were determined by two experimentally based methods that involved the testing of the D-type specimens. Both methods initially used the results from the successive tests of essentially identical simple-span specimens that were made from the same concrete casting. These specimens were subjected to a single transverse load acting at selected positions from one end of the specimen. These tests produced failure modes. For the specimens prestressed with coated-strands, the failure of a particular specimen was classified as one of the following: a flexural failure, a bond failure, a shear failure, a combined flexure and bond failure, a combined flexure and shear failure, or a combined bond and shear failure. For the specimens prestressed with uncoated strands, the failure of a particular specimen was either a flexural, a bond, or a combined flexural and bond failure. For the first method, the strand development length was established as the smallest distance from the transverse load to the closest free end of a specimen for which the failure mode for the specimen involved a flexural component. The testing of the D-type specimens produced convergence to the strand development length or established a range for the strand development length. For the second method, a nondimensional analysis of moment strength versus load position was applied to the test results obtained from the strand development length tests.

In the nondimensionalized study, a critical moment M_{cr} was established as the moment that was induced at the load-point cross section when a particular failure mode (flexure, shear, or bond) occurred in a specimen. This moment was nondimensionalized by dividing it by the nominal moment strength M_n of the PC member that was established from a strain-compatibility analysis. The distance from the transverse load position X to the closest end of a specimen was nondimensionalized by dividing this length by the predicted strand development length L_d that was obtained by applying the AASHTO Specification [1] Eq. (9-32). After establishing a graph of M_{cr}/M_n versus X/L_d for the tests conducted on each size of specimen with either coated or uncoated strands, a sloping straight line was drawn through the appropriate data points and a horizontal line was drawn through the ordinate value of M_{cr}/M_n that was equal to unity. The intersection point of these two straight lines provided the critical X/L_d-value that corresponded to the strand development length of the specimens represented in the graph. Strand development lengths were also calculated by applying the empirical expression obtained from the literature.

A nondimensionalized analysis of the shear strength versus transverse load position for all of the D-type specimens that were prestressed with coated strands was performed. An identical study was also undertaken for all of the D-type specimens that were prestressed with uncoated strands. For a particular strand development length test, the largest induced ultimate shear force V_u that was caused by the ultimate transverse load was nondimensionalized by dividing it by the nominal shear strength V_n of the PC member, as given by the AASHTO Specification [1] Eq. (9-27). The distance X was nondimensionalized by dividing this length by the span length L for the specimen. Graphs of V_u/V_n versus X/L were used to predict whether shear failures of the specimens should have occurred.

The movement of the coated and uncoated strands at the anchorage-end chucks were measured during strand prestressing and during the curing period for the concrete. These measurements were taken to determine if slippage of a strand through a chuck would occur over time. These tests also revealed whether coated and uncoated strands had different anchorage behaviors.

6.2. Conclusions

The conclusions presented in this section have been formulated after analyzing the results for the experimental tests and analytical studies of 3/8-in. diameter, seven-wire, 270-ksi, low-relaxation, grit-impregnated, epoxy-coated strands (coated strands) and 3/8-in. diameter, seven-wire, 270-ksi, low-relaxation bare strands (uncoated strands). The conclusions are applicable for strands that are prestressed to 75% of their minimum tensile strength, which corresponds to a stress of about 202.5 ksi or a force of about 17.2 kips for a 3/8-in. diameter strand, and that are released by cutting with an abrasive grinding wheel (rapid release technique).

1. To prevent concrete splitting during strand detensioning, a 3-in. minimum thickness is required for panels that are prestressed with coated strands and reinforced with smooth-surfaced, epoxy-coated, 6x6-D6xD6 WWF. The strands need to be spaced at 6 in. on center along the midthickness of the panel and have a 3-in. horizontal edge distance from the center of the edge strand to the side of the panel. The WWF, which is placed directly on top of the strands, needs to be cut so that the longitudinal wires of the fabric, which are to be positioned above the transverse wires, occur midway between the strands.
2. A 2 1/2-in. minimum thickness for panels that are prestressed with uncoated strands and reinforced with uncoated 6x6-D6xD6 WWF is adequate to prevent concrete splitting during strand detensioning. The strand locations and WWF configuration must be the same as described in Conclusion No. 1.
3. The average measured transfer lengths for the coated strands in the 6-in. wide by 3-in. thick and 36-in. wide by 3-in. thick T-type specimens were 11.2 and 15.6 in., respectively, when these specimens were prestressed.

4. The average measured transfer lengths for the uncoated strands in the 4-in. wide by 3-in. thick, 6-in. wide by 3-in. thick, and 36-in. wide by 3-in. thick T-type specimens were 35.5, 24.6, and 25.6 in., respectively, when these specimens were prestressed.
5. Except for the 4-in. wide T-type specimens, no substantial changes in the strand transfer length occurred for up to 18 hours after the strands were detensioned.
6. The predicted, coated-strand transfer length that was obtained by applying the ACI Building Code [3] expression, which is also implied by the AASHTO Specification [1], overestimated by about 78% the average of the measured, coated-strand transfer lengths for all sizes of the T-type specimens. If the expressions presented by Cousins et al. [14], and the PCI Guidelines [40] are used, the average of the measured, coated-strand transfer length for all sizes of T-type specimens are overestimated by about 7% and 37%, respectively.
7. The predicted, uncoated-strand transfer length that was obtained by applying the ACI Building Code [3] expression, which is also implied by the AASHTO Specification [1], underestimated by about 15% the average of the measured, uncoated-strand transfer lengths for all sizes of the T-type specimens. If the expressions presented by Cousins et al. [14], Zia and Mostafa [45] and Mitchell et al. [37] are used, the average of the measured, uncoated-strand transfer length for all sizes of T-type specimens are overestimated by about 15%, underestimated by about 31%, and underestimated by about 40%, respectively.
8. When the strands were detensioned with an abrasive grinding wheel, the epoxy coating broke-off from the exposed portion of the strand as the outer wires of the strand unwound

from their original positions. If strand extensions beyond the ends of a PC panel are required, the outer strand wires will need to be rewrapped around the center strand wire, and an epoxy coating will have to be reapplied to any portion of an exposed bare strand in order to maintain the corrosion resistance of the strand.

9. The average measured development lengths of the coated strands in the 4-in. wide by 6-in. thick, 6-in. wide by 6-in. thick, and 36-in. wide by 6-in. thick D-type specimens were 24, 24, and 25 in., respectively.
10. The average measured development lengths of uncoated strands in the 4-in. wide by 6-in. thick, 6-in. wide by 6-in. thick, and 36-in. wide by 6-in. thick D-type specimens were 58, 50, and 47 in., respectively.
11. An interpolation or extrapolation of the results of a nondimensionalized analysis for the coated-strand development length tests showed that the AASHTO Specification [1] Eq. (9-32) overestimated the coated-strand development lengths for the strands in the 4, 6, and 36-in. wide D-type specimens by about 100%.
12. An interpolation or extrapolation of the results of a nondimensionalized analysis for the uncoated strand development length tests showed that the AASHTO Specification [1] Eq. (9-32) underestimated the uncoated-strand development lengths for the strands in the 4, 6, and 36-in. wide D-type specimens by about 43%, 8%, and 3%, respectively.
13. The predicted, coated-strand development lengths established by the expressions given in the AASHTO Specification [1], ACI Building Code [3], and PCI Guidelines [40] were about twice as long as the measured lengths. The measured, coated-strand development lengths were closely predicted by applying the expression given by Cousins et al. [14].

14. The predicted, uncoated-strand development length that was obtained by applying the expression given in the AASHTO Specification [1] and ACI Building Code [3] underestimated the measured uncoated-strand development length for the 4-in. wide by 6-in. thick specimens. However, the predicted length for the 6-in. wide by 6-in. thick and 36-in. wide by 6-in. thick specimens was generally accurate. If the expressions by Cousins et al. [14], Zia and Mostafa [45], and Mitchell et al. [37] are used, the measured, uncoated-strand development lengths are significantly overestimated, slightly overestimated in most instances, and slightly to significantly underestimated, respectively.
15. The measured coated-strand transfer and development lengths were about one-half as long as those measured lengths for uncoated strands.
16. The amount of concrete side cover on an exterior strand affected the uncoated-strand transfer and development lengths but apparently did not affect the coated-strand development length. The influence of concrete side cover on the transfer length for coated strands was inconclusive.
17. The 6-in. spacing used in the 36-in. wide specimens did not appear to affect the transfer or development lengths for either coated or uncoated strands.
18. In order to develop the specified tension force during strand tensioning, the amount of strand movement at the anchor-end chucks for the coated strands was about three times as large as the movement that occurred with uncoated strands.
19. After the strands were pretensioned to 75% of their minimum tensile strength, strand slippage at the anchor-end chucks did not occur with either the coated or uncoated strands.

6.3. Recommendations and Suggested Implementation

Phase I of the research on epoxy-coated strands in composite PC panels has shown that grit-impregnated, epoxy-coated strands can be used to prestress 3-in. thick panels without causing the concrete to split after strand detensioning. The 3-in. minimum panel thickness is one-half of an inch thicker than the present 2 1/2-in. thick PC subdeck panels that are prestressed with uncoated strands. Since the precaster would be providing this additional amount of concrete, the additional cost per cubic foot of precast concrete associated with coated reinforcement compared to uncoated reinforcement of the same size and type would be partially offset, even though the total cost of the bridge deck would probably increase when coated strands and coated WWF are substituted for uncoated strands and uncoated WWF. To address questions related to the economics of using composite bridge decks that contain only epoxy-coated strands, bars, and WWF, preliminary discussions should begin with some of the precast concrete producers to determine ways to reduce the total bridge deck costs to maintain PC panels as a viable alternative to a full-depth, reinforced concrete bridge deck.

6.4. Recommendations for Additional Research

The next logical step for the research on the behavior of epoxy-coated strands in PC panels would be to proceed with the evaluation of the strength and stiffness characteristics for composite bridge deck construction. Phase 2 of the research on epoxy-coated strands in composite PC panels will be proposed and should be conducted to evaluate analytically and experimentally the static load performance of composite slab specimens that contain 3-in. thick PC panels and a 5-in. thick RC topping slab. The PC panels would be prestressed with 3/8-in. diameter, seven-wire, 270-ksi, low-relaxation, grit-impregnated, epoxy-coated strands that are positioned at the middepth of the panel. A layer of 6x6-D6xD6, smooth-surfaced, epoxy-coated WWF would be placed directly on top of the strands. The RC topping slab would contain epoxy-coated reinforcing bars. Companion composite slab specimens containing uncoated strands, WWF, and reinforcing bars would be constructed for comparative purposes.

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